**US Army Corps
of Engineers**
Waterways Experiment
Station

Monitoring Completed Navigation Projects Program

Burns Harbor, Indiana, Monitoring Study

Volume I: Overview of Approach and Results

by   David McGehee, Heidi Moritz, Terri Prickett, Janean Shirley

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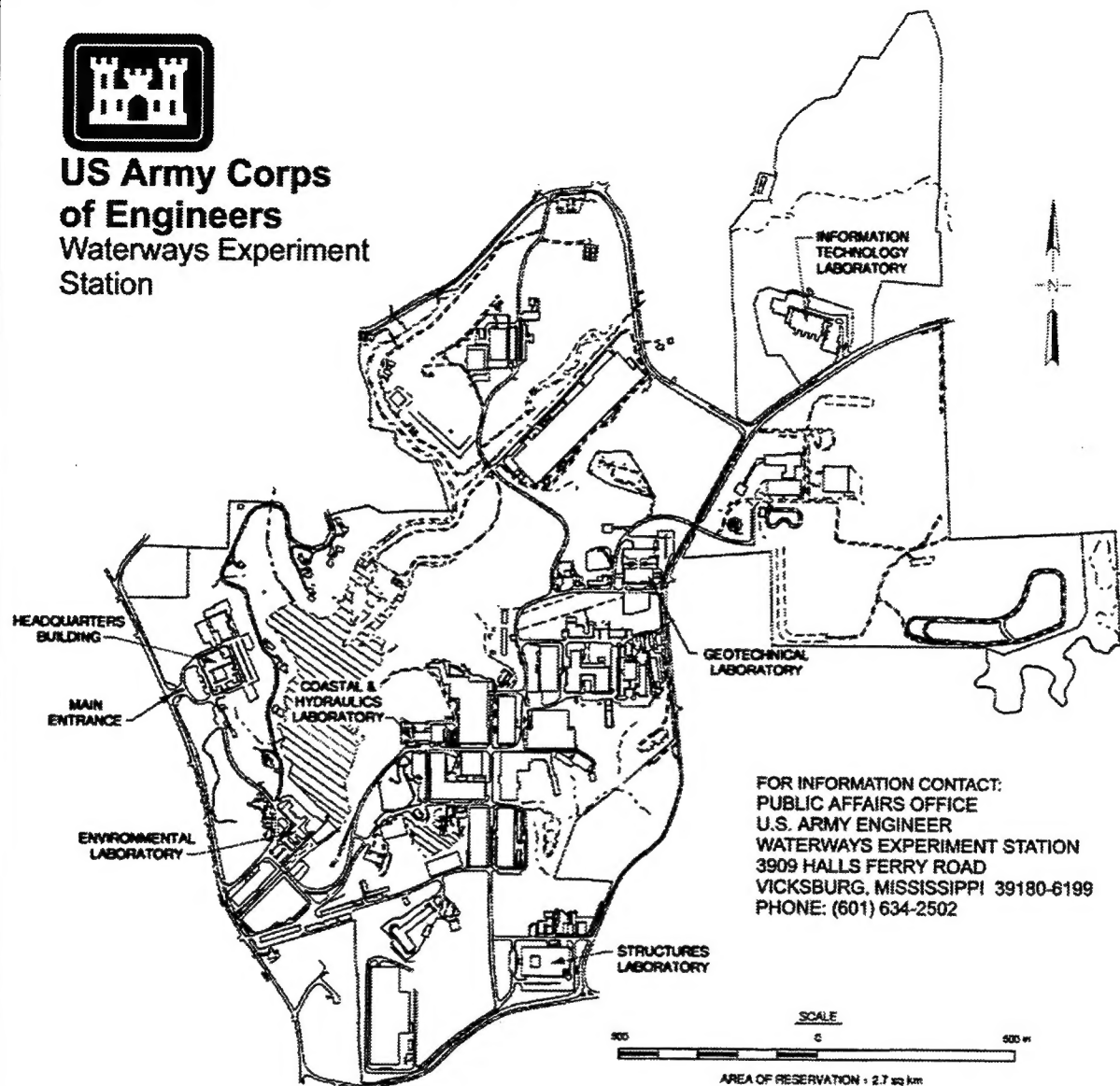3909 Halls Ferry Road
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Final report

**US Army Corps
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STRUCTURES
LABORATORY

SCALE

500   0   500 m

AREA OF RESERVATION : 2.7 sq km

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Preface

This report is being published by the U.S. Army Engineer Waterways Experiment Station (WES) Coastal and Hydraulics Laboratory (CHL). The CHL was formed in October 1996 with the merger of the WES Coastal Engineering Research Center (CERC) and Hydraulics Laboratory. Dr. James R. Houston is the Director of the CHL and Messrs. Richard A. Sager and Charles C. Calhoun, Jr., are Assistant Directors.

This report was prepared by CHL, and is a product of the Monitoring Completed Navigation Projects (MCNP) Program. It represents a joint effort between CERC and the U.S. Army Engineer District, Chicago (NCC). The MCNP Program Manager when the study was initiated was Mr. J. Michael Hemsley. The Program Manager at the conclusion of the study was Ms. Carolyn Holmes. Technical Monitors of the MCNP Program are Messrs. John H. Lockhart, Jr., Charles B. Chesnutt, and Barry W. Holliday.

The Principal Investigator of the Burns Harbor work unit was Mr. David McGehee, CHL. During the course of this study he was supervised by Mr. William Preslan, Chief, Prototype Measurement and Analysis Branch (PMAB), and Mr. Thomas Richardson, Chief, Engineering Development Division (EDD), CHL.

The NCC Principal Investigator during the course of this study was Ms. Heidi Moritz. Ms. Moritz was supervised by Mr. Harwood Herlocker and Mr. Utpal Bhattacharya, Chief, Geotechnical and Coastal Branch, and Mr. Joseph Jacobazzi, Chief, Engineering Division. She received considerable input and assistance from Mr. Hans Moritz, Ms. Joanne Milo, Mr. John Fornek, and Mr. Erik Matthews. NCC personnel were supervised by Mr. Joseph Jacobazzi. During the earlier phase of the study, NCC was ably represented by Messrs. Harry Krampitz and John Panganiban. Additionally, Mr. Charlie Johnson of the U.S. Army Engineer Division, North Central provided valuable historical perspective and continuity through his familiarity with the project.

This report is printed in two volumes. Volume I provides an overview of the monitoring effort, including summaries of elements described in Volume II as well as additional analyses, results, and conclusions. It represents a collaboration between the two principal investigators. Additional analyses and summarization of the extensive "structural" sections were conducted by

Ms. Terri Prickett of PMAB and Ms. Janean Shirley of the WES Information Technology Laboratory (ITL). Ms. Prickett was supervised by Mr. Preslan (PMAB), and Mr. Richardson (EDD). Ms. Shirley was supervised by Ms. Jamie Leach, Chief, Publishing Group, Mr. Bobby Baylot, Chief, Visual Production Center, and Dr. N. Radhakrishnan, Director, ITL.

Volume II contains independently prepared chapters with detailed descriptions of five major elements of the overall study, as outlined below. Technical Editors of Volume II were Mr. McGehee, and Mses. Prickett and Moritz.

Chapter 1: "Project History" was written by Mses. Prickett and Moritz.

Chapter 2: "Results of Analysis of Wave Measurements at Burns Harbor" was written by Mr. McGehee and Dr. Joon Rhee, PMAB.

Chapter 3: "Extremal Analysis of Burns Harbor Hindcast and Measured Wave Data" was written by Dr. Michael Andrew of Jackson, MS. Dr. Andrew is a private consultant and former CERC employee.

Chapter 4: "Evaluation of Breakwater Settlement" was written by Mr. John Andersen, under the supervision of Mr. W. Milton Myers, Chief, Soil Mechanics Branch (GS-S); Mr. Don C. Banks, Chief, Soil and Rock Mechanics Division (GS); and Dr. William Marcuson III, Director, WES Geotechnical Laboratory.

Chapter 5: "Structural Stability Analysis" was written by Ms. Moritz and Mr. Hans Moritz of the U.S. Army Engineer District, Chicago. Mr. Moritz was supervised by Mr. Harwood Herlocker and Mr. Utpal Bhattacharya, Chief, Geotechnical and Coastal Branch, and Mr. Joseph Jacobazzi, Chief, Engineering Division.

Organization and preparation of these reports were coordinated by Ms. Prickett. She received assistance from Mses. Lula Davenport, Joy Brogdon, and Kathy Moore. Other PMAB personnel that provided valuable and patient assistance were Mses. Linda Lillycrop, Wendy Thompson, and Rhonda Lofton. Special appreciation is extended to Dr. Joon Rhee for his illumination of wave theories and Mr. Pat McKinney for his wizardry in programming.

At the time of publication of this report, Dr. Robert W. Whalin was Director of WES. COL Bruce K. Howard, EN, was Commander.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
acres	4,046.873	square meters
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
inches	2.54	centimeters
kilotons	4.185	terajoules
miles (U.S. nautical)	1.852	kilometers
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
square feet	0.09290304	square meters
square yards	0.8361274	square meters
tons (mass)	907.1847	kilograms

1 Introduction

The Monitoring Completed Coastal Projects (MCCP)[1] Program was established by Headquarters, U.S. Army Corps of Engineers (HQUSACE) in 1981 to evaluate the performance of the Corps in planning, design, construction, and operation and maintenance of selected Civil Works coastal projects. The MCCP is funded by the Operations and Maintenance (O&M) Division, HQUSACE, and managed by the U.S. Army Engineer Waterways Experiment Station (WES) Coastal and Hydraulics Laboratory (CHL).[2] Oversight is provided by a Field Review Group composed of representatives of Corps Divisions with coastal interests, Program Monitors from HQUSACE, and the Coastal Engineering Research Board. The program's objective is to acquire information through intensive monitoring of coastal projects in an effort to improve the following:

　a. Project purpose and attainment.

　b. Design procedures.

　c. Construction methods.

　d. Operation and maintenance techniques.

Potential projects are nominated by coastal Districts and selected for monitoring during an annual Program Review attended by the Field Review Group. Selection is based on the potential for improving engineering methods for application at the study site and other project sites.

Burns Harbor was nominated for inclusion in the MCCP by the U.S. Army Engineer District, Chicago (NCC). The original nomination from NCC is provided in Volume II, Appendix 1A. Burns Harbor was approved for monitoring in FY85 because it met both generic and site-specific selection criteria.

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[1] The MCCP Program was renamed the Monitoring Completed Navigation Projects (MCNP) Program in October 1996. In this report, however, the program will be referred to as MCCP.
[2] The CHL was formed in October 1996 with the merger of the WES Coastal Engineering Research Center (CERC) and Hydraulics Laboratory. For historical purposes, however, any reference to the WES laboratory will be cited as CERC.

criteria. The monitoring effort and its documentation were a joint effort of CERC and the NCC.

One of the criteria for selecting a project for monitoring should be availability of adequate background information. A considerable body of documentation on the project existed in the NCC files. Records spanned 1943 proposals for harbor siting to the most recent 1992 surveys. Formats ranging from published reports and memos to drawings, photographs, videotape, and computer files were distributed in a dozen locations. The volume of these documents made sorting, reviewing, analyzing, summarizing, and integrating their content into coherent results a major task of the study. The index of documents and drawings compiled for the study is found in Appendix A of this volume. Nevertheless, this information was not sufficient to answer all questions that arose. Some gaps were filled through interviews of principals when available, but complete resolution of many important issues was not possible.

This report is divided into two volumes. Volume I provides an overview, containing background information, a description of the monitoring effort, results of analyses, conclusions, and appendices of supporting information and records. Its purpose is to synopsize the study and state the important conclusions for the general-interest reader. Volume II contains the detailed procedures and analyses for the major elements of the study used to obtain the results in Volume I; i.e., project historical review, wave data analysis, wave extremal analysis, foundation analysis, and structural stability analyses. Additional project materials that were compiled but are not suitable for this report, such as computer files of wave energy spectra, photographs, videotapes, sonograms, etc., will be maintained at CERC and NCC for future reference.

Chapter 1 of Volume I is the introduction. Chapter 2 describes the federal project at Burns Harbor, including general characteristics of the site, the harbor plan, and the breakwater. Chapter 3 is the history of the planning, design, construction, and operation of the project. This history is the "product" of one of the major elements of the monitoring plan, and could be considered the first of the results presented in Chapter 5, "Monitoring Results," but is presented separately due to its size and relevance to the subsequent sections of the report. Chapter 4 describes the monitoring plan that evolved in an attempt to address both site-specific and general issues. Chapter 5 presents the monitoring results. Chapter 6 discusses the results and presents the study's conclusion. Chapter 6 also presents "lessons learned" for application to Burns Harbor; for general application to coastal design, maintenance, and operations; and - equally important - for conducting successful monitoring studies. Chapter 7 presents a brief summary of the entire study.

2 Project Description

Site Description

Burns Harbor, IN, is located on the southern end of Lake Michigan (Figure 1). The configuration of Lake Michigan exposes the harbor to significant wave energy from the northern quadrant. Maximum fetch is about 300 miles,[1] to the north. Extreme weather usually occurs during passage of cold fronts associated with extratropical cyclones. Water levels fluctuate seasonally on the order of a meter due to regional precipitation patterns and near a meter during storms from wind setup/setdown.

Active glaciation in the Pleistocene Era caused radical variation in lake levels and a wide diversity in depositional patterns around the Great Lakes. The result in the vicinity of Burns Harbor is an inhomogeneous distribution of sand and gravel lenses in a matrix of both stiff and soft clays. Figure 2 is a simplified cross section of the lakebed within the harbor showing typical distribution of the the three predominant soil types. The natural shoreline consists of high dunes of medium sand. The lake bottom is covered with silty sand; slopes are on the order of 1:100 offshore of the harbor.

Harbor Layout

The federal project consists of an L-shaped breakwater with a western arm 1,200 ft long and a northern arm 4,640 ft long (Figure 3). A cellular sheet-pile extension connects the western arm to the shore. The depth of the lake-side toe of the northern arm ranges from -30 ft to -41 ft low water datum (LWD). The authorized project depth is 30 ft in the entrance channel, and 28 ft in the harbor, though actual depths are typically more.

The interior perimeter of the harbor has both riprap revetment and vertical steel sheet-pile sections. The east harbor arm is bordered by the Bethlehem Steel plant; the west harbor arm flanks the Midwest Steel mill; and the central

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[1] A table of factors for converting non-SI units of measurement to SI units is presented on page ix.
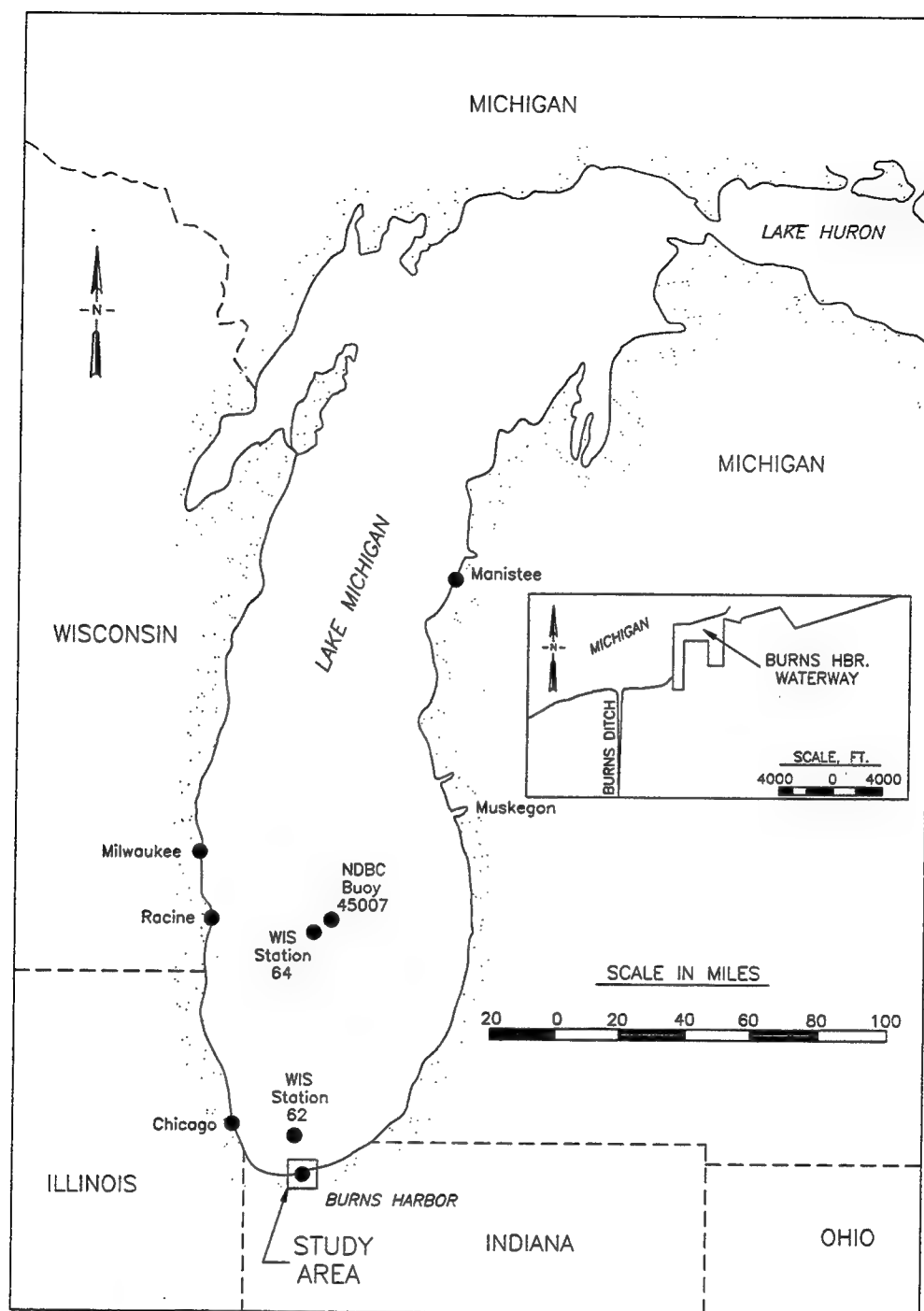
Figure 1. Location of Burns Harbor, IN
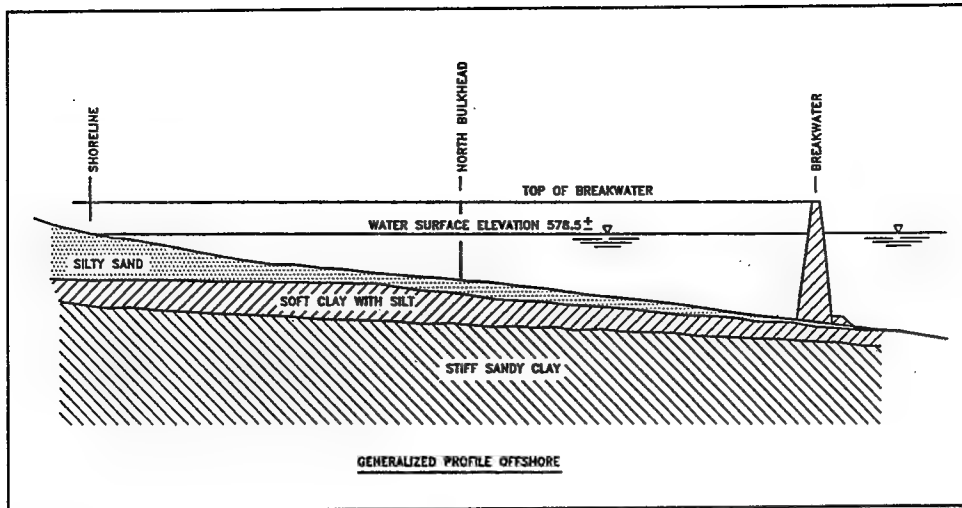
Figure 2. Pre-construction geologic cross section of lakebed
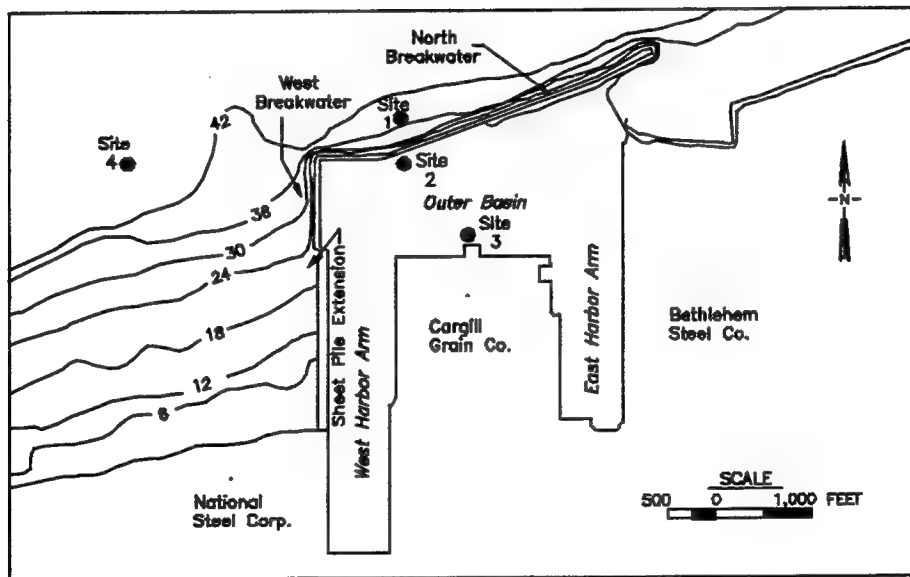
Figure 3. Plan of Burns Harbor with wave gage locations

portion of the harbor, or north wharf, is occupied by the Cargill Grain Co. facility. The Cargill grain dock is a rectangular sheet-pile quay projecting outward from the adjacent riprap-covered fill.

Breakwater

The breakwater is a multilayer rubble-mound structure with two layers of random-placed Bedford limestone armor (W stone). Figure 4 is a typical cross section for the northern arm. Design crest elevation is +14 ft LWD. Side slopes on both lake- and harborsides are 1:1.5. The core stone is 5-90 lb and projects about 12 ft beyond the W/10 stone to form a bedding layer for the armor. A sand core forms the lowest layer, presumably contiguous with sand backfill placed in certain locations to prepare the foundation, as described later in the report.

The parallelipiped, cut stone armor units, which range from 10 to 16 tons on the trunk and from 15 to 20 tons on the head, are typical for coastal structures in the Great Lakes, but the two-layer random placement was unusual at the time of its design. A high core design with laid-up placement of a single layer of armor is more typical. Burns Harbor was the first breakwater with this style of armor placement built in the Great Lakes (Figure 5).
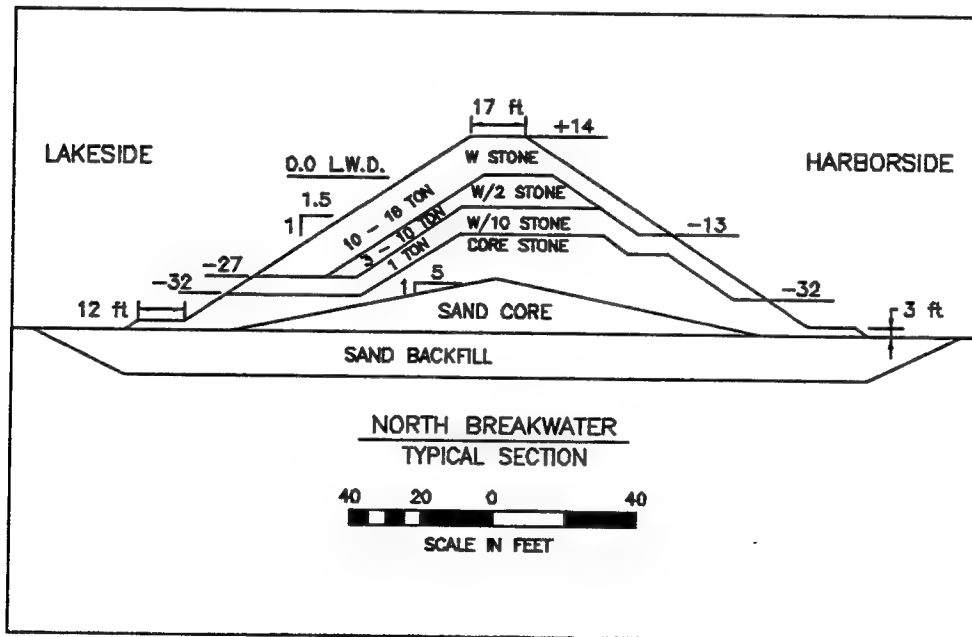
Figure 4. Typical breakwater cross section

a. Lakeside view during construction
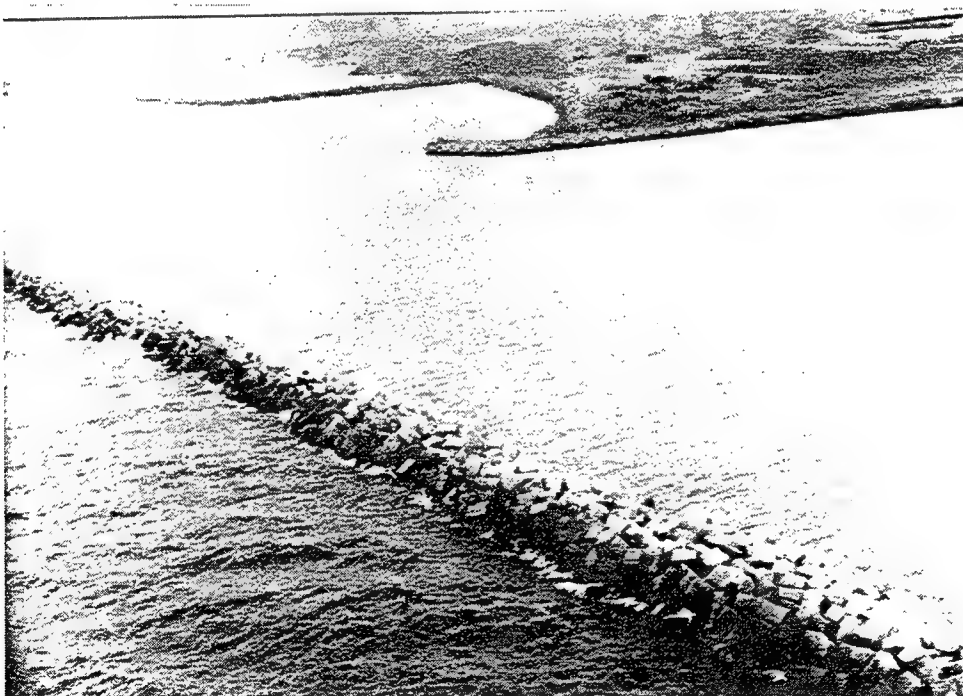
b. 1987 lakeside view

Figure 5. Views of armor layer

3   Project History

Planning

A harbor was proposed to accommodate industrial expansion in northern Indiana as early as 1931. The NCC initiated a survey report to study the benefits in 1951, and submitted the Great Lakes Harbor Study Interim Report in 1962, which evaluated several options for Burns Harbor. The study considered various alternatives for a breakwater design, including the traditional laid-up armor over a high core and cellular steel sheetpiling. The NCC proposed a harbor covering 225 acres, protected by a breakwater in 43 ft of water. Though Congress authorized the project with the Rivers and Harbor Act of 1965, federal funds were not provided at that time. Instead, funding was undertaken by the state of Indiana for the entire harbor development with the provision that the federal government would reimburse the state for the federal portion.

Design History

Design sequence

Responsibility for detailed design of the harbor was assigned to Sverdrup and Parcel and Associates (SPA), with the assistance of consultants from WES, the University of Florida (UF), and the Navy Department. The Corps of Engineers NCC, North Central Division, and HQUSACE formally reviewed the breakwater design. The major elements to be designed were the harbor plan, the breakwater cross section, and the foundation.

SPA developed the initial harbor plan in the early 1960's. In 1964, the UF tested the initial SPA harbor plan using a three-dimensional physical model (Indiana Port Commission 1966 a and c)[1]. By 1965, the UF-modified harbor plan was augmented with a preliminary breakwater cross section. This design was reviewed and modified during two design conferences in 1965. A

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[1] The reference list at the end of this volume contains all references in Volumes I and II of this report.

two-dimensional physical model test was conducted at WES in 1966 to optimize the final breakwater cross section. Foundation design was completed by March 1966.

Harbor plan design

Approach. The final harbor layout resulted from a three-dimensional (3-D) hydraulic model study conducted by the UF. The main objective of the 1:150 scale model was to develop plans that minimized the following:

a. Navigation risks at the harbor entrance.

b. Wave transmission through the entrance.

c. Reflection of wave energy coming through the entrance by harbor boundaries.

d. Harbor seiching.

e. Adverse effects from wave overtopping and reflection, shoaling, and ice.

The basic harbor plan tested in the model study is essentially the current configuration. This layout is reduced in area from the original Corps plan to save costs, principally by placing the breakwater in shallower water. Alternative tests focused on design of the harbor entrance. All dimensions in the following discussion refer to the prototype, or scaled, dimensions.

Harbor representation. The UF model represented the north breakwater as an impermeable barrier with an external slope of 1:2. Interior harbor boundaries were vertical, impermeable walls. The north wharf did not contain the projecting quay. Monochromatic wind waves and long waves were generated with a movable flutter-type wave maker (limited tests were done using multiple-period wave trains between 5.4 and 9.8 sec, but the results "... showed no evidence of harmful effects...," so regular waves were used for testing). A model ore carrier weighing 27,540 tons and 626 ft in length was placed at various locations. Details on its mooring arrangement were not provided. Currents and ship response were visually observed.

Wind waves. Design wind waves for the 3-D model study were obtained by hindcasting, using wind data from Duneland Observatory at Ogden Dunes, IN, 2.5 miles west of the harbor, from April 1956 through March 1959, and from June 1961 through May 1963. Only winds with directions from northeast through northwest were considered. Winds from December through March were rejected on the assumption that lake ice would prevent wave formation on the margins of the lake.

Three deepwater design waves were designated in the UF study. Design wave 1 was based on the longest available fetch (300 miles from 8 deg 30' true) and the average wind speed exceeding 20 mph in the northwest quadrant, or 25 mph. The resulting hindcast deepwater significant wave was 10 ft high at a 9-sec period. Design wave 2 was based on the most direct exposure (northeast) and the average of winds exceeding 10 mph from this quadrant, resulting in a hindcast deepwater significant wave of 3 ft at 5 sec. Finally, design wave 3 was the most frequently occurring wave: 2 ft at 5 sec from the north. All three were considered conservative. To obtain the angle of incidence at the harbor, ray diagrams were used to refract the waves from a water depth of 50 to 40 ft, the presumed depth of the structure. Presumably, refraction between deep water and 50 ft was ignored.

Design waves 2 and 3 were not considered reproducible at this scale because surface tension influenced their behavior at their short (model scale) wavelength. However, design waves 1 and 2 were used to test the effect of the length and angle of the eastern end of the north breakwater. The incident waves modeled for harbor response were: 8.5 ft at 5.4 sec; 7.5 ft at 7.4 sec; and 6.2 ft at 9.8 sec. The report is not clear on why these selections were different from the design wave, except for the earlier mention of surface tension effects.

Long waves. The long waves of particular interest in the study were those with periods below 3 min because the resonant periods of the vessels assumed to frequent the harbor (vessels between 2,000 and 50,000 tons displacement) are generally less than 2 min. This range covered all fundamental and higher harmonics of the basin. Long waves of this type were considered rare in the lake, though a record-setting storm in 1963 causing oscillations on the order of 1/2 ft was cited. Surge periods tested ranged from 50 sec to 2.5 min. Incident amplitudes ranged from 1.5 to 3 ft, which were considered exaggerated by about a factor of 10 over expected values. This was done to permit measurement with existing, parallel-wire model wave gages. Lake seiches with periods on the order of 15 to 70 min were assumed to produce currents through the harbor entrance, but not induce significant ship response.

Littoral transport. Net littoral sediment drift was assumed to be toward the west on the order of 27,000 yd^3/year (based on adjacent accretion patterns, dredging patterns of nearby harbors, etc.). This was not considered a potential problem with respect to shoaling of the channel because the planned landfill to the east of the entrance served as an effective littoral barrier, and the planned entrance depth of 34 ft was 4 ft deeper than required.

Lake water levels. The water level used in the model was 3.1 ft LWD, based on the long-term average for Lake Michigan of 2.1 ft plus an assumed surge of 1 ft due to a storm of "... moderate intensity in this area," with a frequency of occurrence of once a month.

Results and recommendations. Tests were conducted to optimize the length, orientation, head geometry, height, and slope of the north breakwater

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with regard to response to wind waves at the entrance and interior wharfs. The recommended length of the north breakwater tip (that portion projecting eastward of the eastern boundary of the harbor basin) was 700 ft. One of the principal results of the model study was the recommended harbor entrance design: a 10-deg rotation of the eastern tip of the breakwater, and a curved, revetted wave absorber landward of the entrance channel, offset landward by 645 ft from the outer limit wall to the east. In addition, a "streamlined," rounded head configuration with a shallow (about 1:3) lakeward slope was suggested.

The slope on the remainder of the breakwater was considered satisfactory at 1:2, principally in regard to reflection-caused problems to small craft navigating near the structure. Of note is the suggestion to place rock on the outer slope so as to "...display maximum degree of permeability and stability simultaneously..." with its longest axis perpendicular to the breakwater. A crest elevation of +12 ft was considered adequate to prevent overtopping. However, the smooth impermeable walls were not expected to accurately simulate wave reflection from or transmission through and over the north breakwater.

Wind wave heights in the harbor typically were 15 to 25 percent of the incident height, with the exception of locally higher values near corners. Nine-sec waves (which were considered infrequent) tended to produce a standing oscillation with an amplitude on the order of 10 percent of the incident amplitude. Wind waves did not measurably displace the model vessel.

The model detected practically no resonance in the east-west direction. Seiche in the north-south direction in the east and west slips was observed at the second (110-120 sec), third (70-80 sec), and fourth (50-60 sec) harmonics at amplification factors from 150 to 200 percent. Vessel motion at the north wharf was negligible. In the east and west slips, north-south vessel excursions ranged from 3 to 9 ft for an incident amplitude of 0.3 ft. This was considered to be of little consequence. Modifying the harbor geometry by straightening the bend at the west end of the north breakwater did not affect harbor oscillations.

Deposition in the entrance was considered "... of no immediate concern" due to the existing depths. Erosional effects on the downdrift shoreline were anticipated at about 27,000 yd³/yr. Bypassing or construction of protective structures for the beach to the west were suggested. It was anticipated that ice would jam the entrance, but this was considered to be infrequent during the navigation season.

The Lake Carriers Association accepted the reduced area and entrance design as suitable from a navigation standpoint. The alignment of the break-water, particularly the 10-deg rotation in the easterly section, generated questions and discussion since it would complicate horizontal control during construction, but it was eventually accepted by the Corps.

Preliminary cross section

A preliminary design was developed by SPA, with rubble-mound structures now comprising the main breakwater elements. The armor units were cut Bedford limestone. Using the same wind data set described for the 3-D model, SPA selected a design wave of 11.5 ft. The Hudson formula was used, with an assumed stability coefficient K_D of 3.5 and a slope of 1:1.5 to specify an armor unit of 8 to 9 tons. The crest elevation was specified as 1.2 times the design wave, or 14 ft. Sand was specified for the core to take advantage of other port development requiring excavation of the dunes occurring on shore.

Design conferences

Two conferences were held in 1965 to review the preliminary design for Burns Harbor. The minutes of those design conferences are given in Volume II, Appendix 1A. Participants included representatives of NCC, SPA, the Indiana Port Commission, and their design consultants: Mr. Ayers from the Navy Department, Dr. Per Bruun from UF, and Robert Hudson from WES. Topics covered selection of the design wave and water level, design of the cross section (armor layer type and placement, underlayer dimensions, core height, etc.), design of interior harbor surfaces, occurrence of ice, navigation issues, and economics.

Much discussion concerned selection of the nonbreaking design wave for the project. Since the principal design formula at the time, e.g., the Hudson formula, used wave height as the only input wave parameter, most discussion focused only on the design wave height. Mr. Hudson recommended 16.5 ft, based on a Beach Erosion Board study predicting a return interval of 25 years for a deepwater wave of 18 ft near Chicago in Lake Michigan. Dr. Bruun, using the Bretschneider method for the available wind data, recommended a 10.5-ft design wave, which should neither overtop nor damage the preliminary cross section. Mr. Ayers suggested 12 ft. The disagreement centered on two parameters, wind speed and fetch, used to hindcast wave conditions using the Bretschneider method. The available wind record from Ogden Dunes spanned about 10 years, which some participants considered barely adequate to represent the climate. Selection of the design wind conditions was strongly affected by the assumption by some of the participants of an "ice season" in the winter that would prevent wave formation during the higher wind events. In addition, some participants argued that the measured fetch of 300 miles could be reduced to an effective fetch of half this distance due to the limited width of the lake. A compromise fetch of 225 miles for a wind speed of 35 mph was eventually agreed upon. The resulting design wave height was 13 ft.

A design that prevented overtopping for a 13-ft wave still required a significant structure in that depth of water. In the end, the conferees decided that a cross section which met two design criteria "... from an economic viewpoint" should be designed. The structure should (a) prevent overtopping for an

11-ft incident wave, and (b) remain stable, while allowing overtopping, for a 13-ft incident wave. The rationale for the two design waves is not detailed in the minutes, but presumably it was associated with the cost of making the structure crest high enough to prevent overtopping by the 13-ft design wave. The suggestion was made that the crest could be supplemented with a concrete cap at a later date if overtopping appeared excessive.

Similarly, economic motives are apparent in the discussions of the structure slope. Though Mr. Hudson recommended a slope of 1:2 to mitigate potential impacts on small vessels from wave reflection, it was held to 1:1.5 to limit overall structure size.

Another topic of discussion was the effect of wave transmission from overtopping on the interior of the harbor. Mr. Hudson felt the small size and rectangular plan would make energy trapped in the harbor "problematical." The preliminary design mentions the possibility of overtopping when considering a crest elevation sufficient that waves in the harbor "... not exceed allowable limits." However, the record contains no quantifiable recommendation for the allowable transmitted wave height in the harbor.

Conferees also discussed the attributes of the Bedford limestone. Though locally available, it had a low specific weight of 145 pcf, and was quarried in rectangular blocks. The shape was desirable when used in laid-up placement for a single layer, but the ability to construct a stable two-layer slope was questioned.

Final cross section design

Approach. The design conferences resulted in two candidate designs: one using a tribar armor layer, and one using cut limestone. The final breakwater cross section was optimized using a two-dimensional (2-D) 1:35 scale physical model. The stated objectives of the study were: (a) to develop a stable design that would not be unusually difficult to construct, and at the same time would make optimum use of the different sizes of stone from the quarry; and (b) to determine the heights of waves on the harborside of the breakwater resulting from the transmission of wave energy over and through the structure. Tests were conducted on both the rectangular, cut-limestone blocks and concrete tribars. Specifically, "...it was desired to determine the feasibility of placing this type of stone in a random manner in such a way as to obtain the desired stability."

By the time of this study, it was accepted that the design would inevitably allow some wave energy over and through the breakwater, contrary to the assumption in the 3-D model study. The limited harbor space precluded revetted interior walls as wave absorbers in most locations, so it was necessary to keep the transmitted wave height below some value. That threshold value, which should be a criteria for determining the performance of the project, was unfortunately never explicitly stated.

Test conditions. A 13-ft wave height was selected to represent the proto-type design wave. Tests were conducted on a range of wave heights from 5 to 20 ft for three wave periods of 7, 9, and 11 sec. Two water levels, 0 and 4 ft LWD, provided water depths at the toe of 43 and 47 ft, respectively.

The tests were conducted at the WES flume (119 ft long by 5 ft wide by 4 ft deep) using a regular, plunger-type wave generator. A concrete floor extended 72 ft (model dimension) from the toe of the breakwater at a slope of 1:100. The procedure for measuring the incident wave heights was not speci-fied. It will be assumed for the following discussion that adequate measures were taken to avoid inclusion of reflected energy in this parameter. Transmit-ted wave heights were measured at two locations behind the breakwater, corre-sponding to the horizontal distances of $L/2$ and L from the structure center line, where L is the wavelength measured in the depth of water at the toe. A total of eight plans were modeled; one with two layers of 5-ton tribar armor, and with seven different configurations of 10- to 16-ton rectangular limestone armor.

Results. The preliminary SPA design (Plan 2) was found to be stable for 11-sec, 13-ft waves, but damaged by 15-ft waves. Damage occurred on both the lake- and harborsides of the structure. The optimum plan (number 8) used two layers of 10- to 16-ton random-placed limestone block armor from the crown down to -27 ft on the lakeside, and one layer of armor from +3 ft to -13 ft on the harborside. Though two armor stones were displaced from the lakeside during the test, this design was considered stable for 11-sec, 15- to 18-ft waves. For a still-water level of +4 ft, the maximum transmitted wave height for the 13-ft incident design wave was about 3 ft. Figure 6 is a repro-duction of the measured transmitted wave data from the study.

It is important to note that the optimum plan allowed more transmitted energy than the preliminary plan for incident waves larger than the 13-ft design wave. This allowance was attributed to the increase in porosity from the use of armor on the harborside. By selecting Plan 8 as optimum, improved stability was chosen at the expense of increased transmission.

An instructive result of the study is that from Plan 5, which used uniform placement of the rectangular armor. Stability was improved, but at the expense of using 25 percent more armor. However, total transmission increased by about 50 percent. Even though this design was less permeable, overtopping of the smooth surface increased dramatically.

Foundation

SPA explored the foundation through testing and classification of 14 bor-ings up to 50 ft deep along the planned breakwater alignment. Standard Pene-tration Test results and Atterberg Limits were obtained for all boring sites. Consolidation tests were performed on 12 sub-samples, but information con-cerning the rate of consolidation was not provided. Design of the foundation
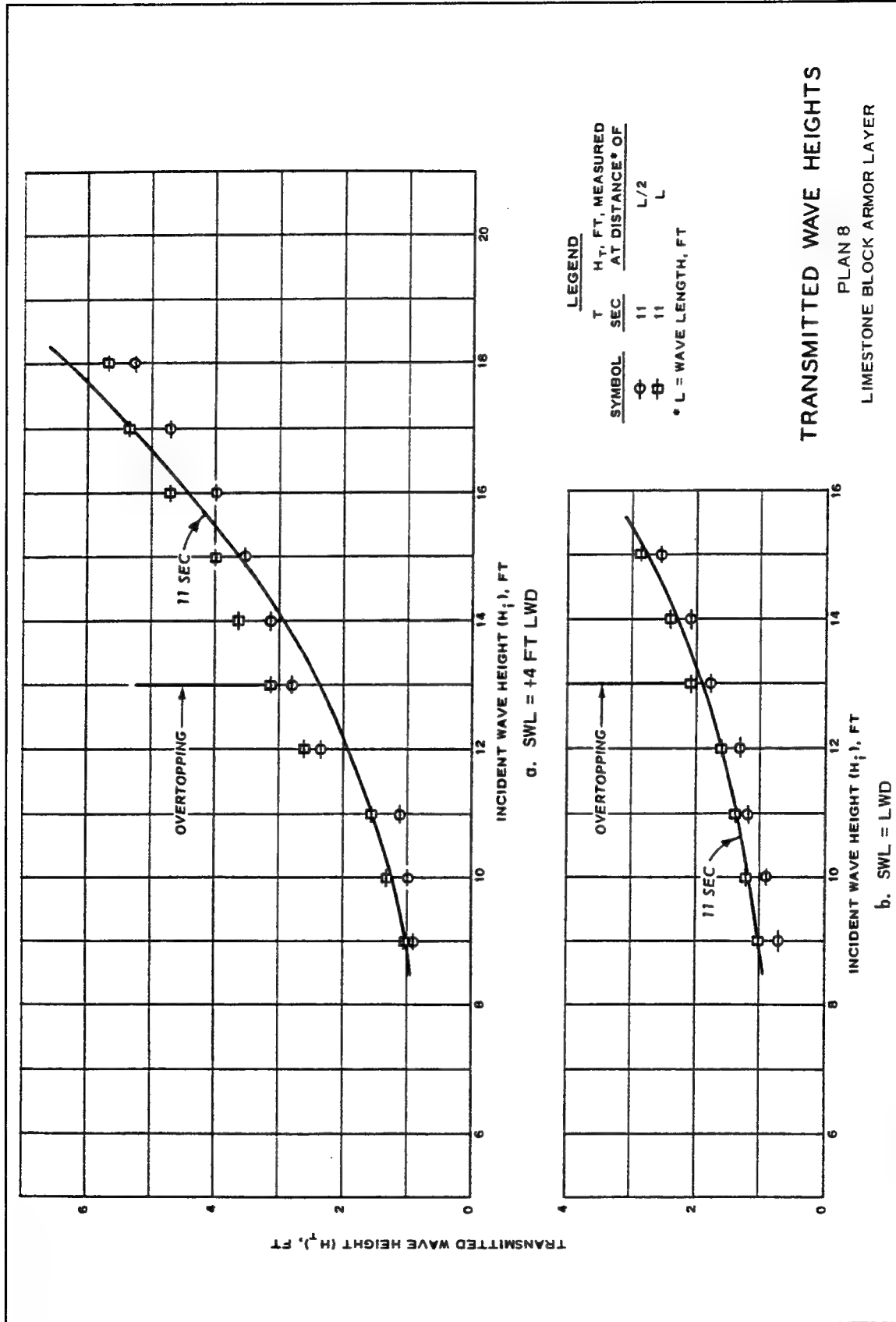
Figure 6. Transmitted wave heights from 1966 2-D study

was challenging because of the variability in the material properties of the clays, sand, and gravel underlaying the lake bottom, and their distribution throughout the harbor area. The uppermost layer of fine and medium sand ranges from 0 to 8 ft thick. Below the sand is a layer of soft, silty clay with some gravel, ranging from 0 to 20 ft thick. Lowermost is a glacial till consisting of stiff silty clay, occasionally mixed with sand and gravel, extending to the maximum boring depths.

In the initial plan submitted in 1965, the only foundation preparation proposed was a 5-ft sand blanket placed on the lakebed. The next year, a report prepared by another consultant predicted variable settlement on the order of 1 to 2 ft (up to a maximum of 2.5 ft) would occur from consolidation of the upper 13 ft of soil. Consolidation of layers below 13 ft was assumed to be negligible. To prevent this settlement, the foundation was prepared by excavating the clay layers to depths varying from 0 to 20 ft, as determined from analysis of boring logs along the structure's center line, and back-filling the trench with sand prior to core placement. Figure 7 illustrates the depth of excavation and the elevation of the sand backfill for each 100-ft section of the breakwater. Station numbers begin at zero at the eastern tip of the breakwater.

Construction History

Breakwater construction commenced on 2 June 1966. The first vessel unloaded cargo in the harbor on 11 September 1969. Construction progressed simultaneously in overlapping stages; excavation of the lakebed to the design depth was the first step, followed by backfilling with sand from the dunes being leveled for construction of port facilities. No information on the placement method for the sand core is available. Bedding stone was placed over the sand by conveyor belt. Stone layers were dumped or, for the armor stone, individually placed by crane.

Construction started on the west end of the north breakwater and proceeded eastward; then the western arm was completed. Figure 8 represents the sequence of excavation and sand backfill operations based on contractor progress reports. Figures 9 and 10 are photographs of placement of the core stone and armor units, respectively, during construction. Stakes used for position control are visible. Figure 11 is a typical cross section from the as-built survey conducted by the construction contractor, and the same section obtained in a 1975 condition survey.

Environmental Loading History

Wave conditions

Wave loading over the life of the structure was required to compare predicted and observed damages in the structure stability effort. The wave history
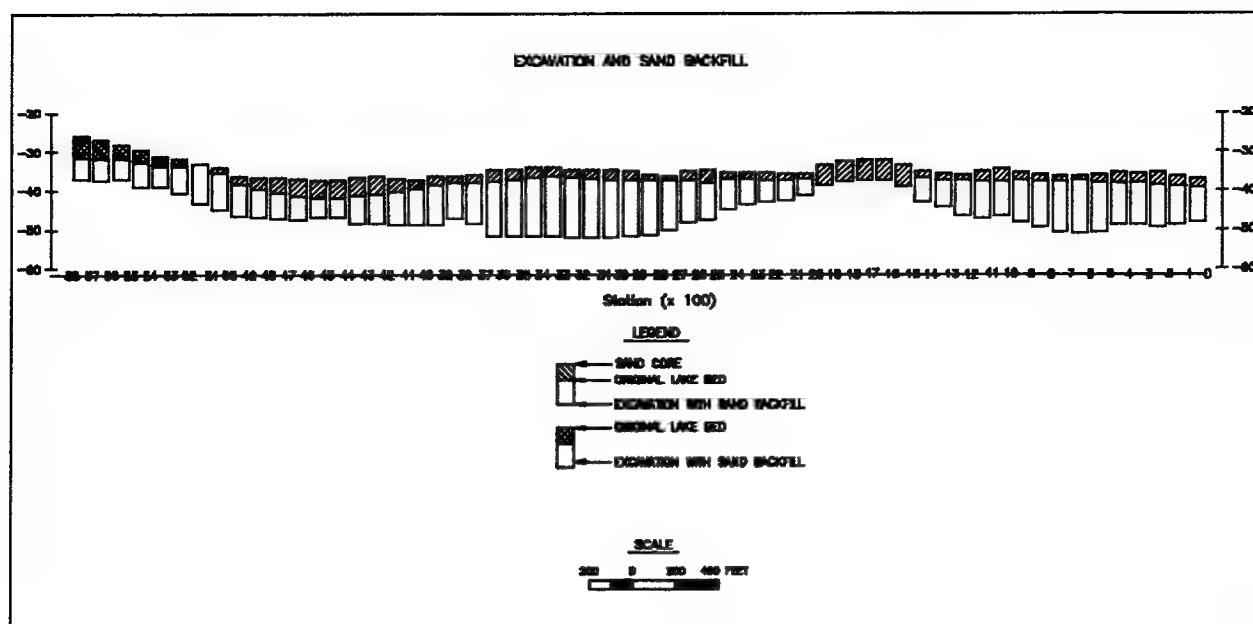
Figure 7. Foundation excavation and backfill design

was obtained from a hindcast conducted by the Wave Information Study
(WIS), a CERC program that numerically calculates wind vectors and
2-D wave spectra every 3 hr at selected locations (called stations) around the
United States from historical meteorological input. The latest available update
of the hindcast for Lake Michigan, WIS Report 24, utilizes 32 years of hind-
cast data, from 1956 to 1987. The Lake Michigan station nearest to Burns
Harbor is No. 62, located approximately 10 n.m. north (Figure 1). A subset of
storm events was selected for transformation to the shallower site. The trans-
formation technique requires the following parameters: peak period, water
depth, and wind speed. The theory assumes fully saturated seas and produces
the energy-based significant wave height for a given water depth.

A storm was defined as occurrence of winds over 20 mph from the north-
ern quadrant (315 to 45 deg, true) for a duration over 9 hr. The storm data set
so defined contained 384 events for station 62. Figure 12 shows the number
of hindcast storms that occurred by month and year. As expected, the stormy
season is the winter, with February the stormiest month. The interannual vari-
ation is irregular; for example, the decade of the 70's appears particularly
stormy. The storms were ordered by hindcast significant wave height, and the
32 maximum events selected for transformation.

Of the 32 events, 7 occurred between 1985 and 1987 when measured wave
data from the gages at Burns Harbor were available for comparison. Table 1
shows the wave heights and periods from the WIS data at Station 62, the
transformed hindcast data, and the measured data for those events. With the
exception of the January 1987 event, transformed wave heights agreed within
0.2 m. Wave period, which is conserved in the transformation process,
showed poorer agreement with measurements, with an average difference of

Progress Report #	5, 6	7	8	9	10	11	12	13	14	15	16
Year	1966		1967						1968	—	
Month	Aug	Dec	Jan-Mar	April	June	Aug	Oct	Nov	Dec-Mar	April	June

Station Number (x 100)

Legend:
- Excavated Clay
- Sand Backfill
- Partially excavated or backfilled

	5, 6	7	8	9	10	11	12	13	14	15	16
Excavated Clay (cu yd)	NA	168,805	No Activity	NA	50,000	81,000	47,000	NA	No Activity	NA	Total NA
Sand Backfill (cu yd)	NA	129,740	No Activity	NA	106,700	58,500	87,800	NA	No Activity	NA	NA

NA = Not Available

Figure 8. Excavation and backfill sequence

Figure 9. Corestone placement by conveyor
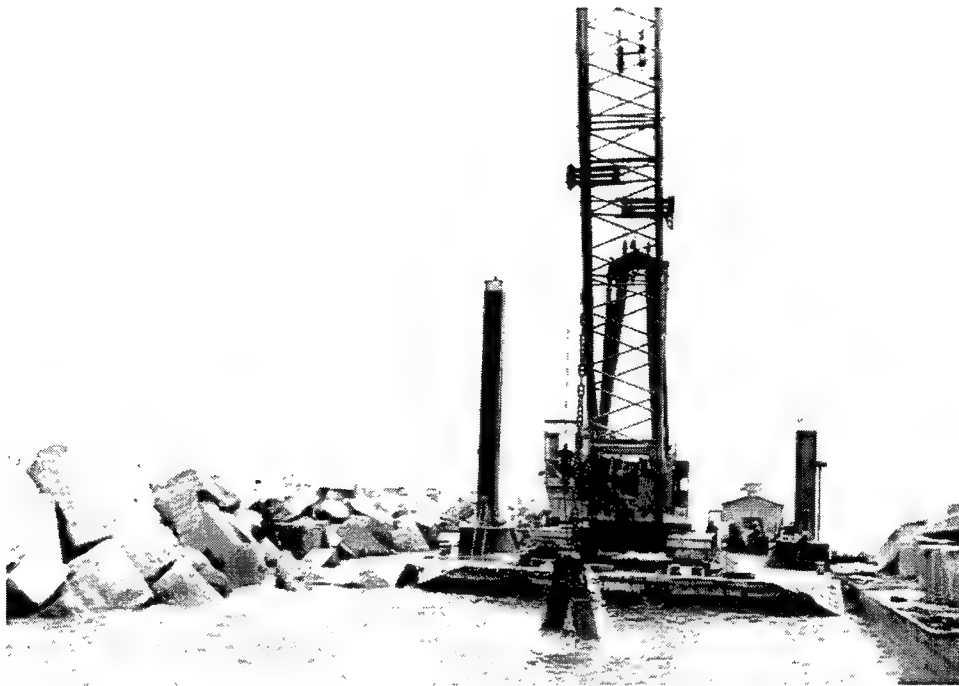
Figure 10. Armor stone placement during construction
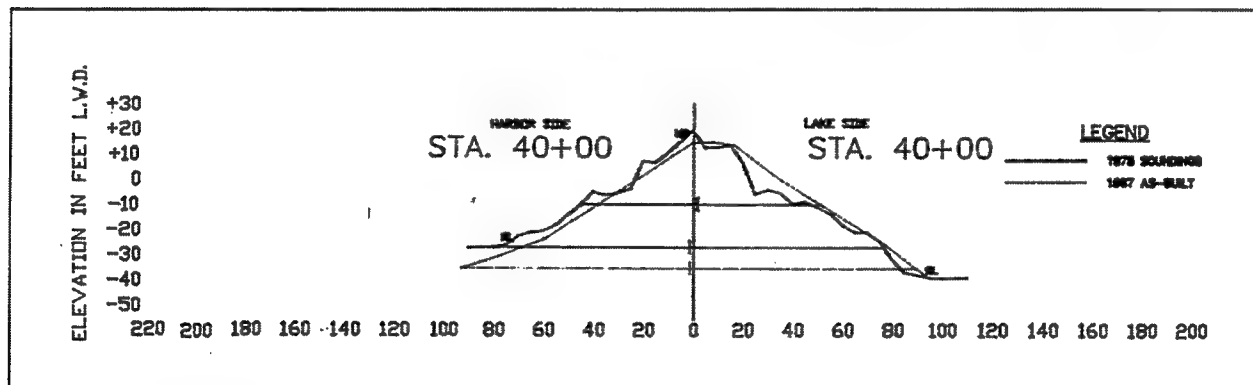
Figure 11. Typical as-built cross section

2.4 sec. Since wave period is an important parameter in performing the wave transformation, a 2-sec upward adjustment was made to the WIS storm subset (based on this apparent bias), prior to transformation for use in the structure stability analysis. No adjustments were made to the transformed wave heights for the stability analysis.

Water levels

Water levels are available from the National Oceanic and Atmospheric Administration (NOAA) lake level gage (No. 7044) at Calumet Harbor located

Table 1
Hindcast and Measured Storm Events

Date of Event	Measured Data		WIS Station 62		Transformed WIS	
	H^1	T^2	H	T	H	T(adj)
24 Dec 85	2.5	9.9	2.4	6.7	2.7	8.7
13 Jan 86	2.7	9.9	2.5	6.3	2.8	8.3
23 Jan 87	2.3	9.2	2.6	6.7	2.8	8.7
08 Feb 87	4.3	11.6	5.4	10.0	4.5	12.0
09 Mar 87	3.2	10.7	3.1	7.1	3.2	9.1
05 Apr 87	2.3	7.1	2.7	5.9	2.4	7.9
04 Dec 87	2.6	8.5	1.8	6.3	2.5	8.3

[1] H = Significant wave height, m.
[2] T = Peak period, sec.

approximately 20 miles to the west of Burns Harbor. The gage is a float-in-stilling-well type that records analog output for postprocessing. Mean water levels to LWD are calculated over each consecutive 6-min interval, and reports of monthly and annual statistics are published. Figure 13 is a time series plot of the annual average, the maximum monthly average, and the minimum monthly average lake levels for each year from the Calumet Harbor gage since 1903. Coincidentally, the extreme excursions for the monthly average lake level over the century cover the period from initial design of the harbor (-1.45 ft in March 1964) to the beginning of the MCCP study (+5.05 ft) in June 1986. The 1986 annual average was 4.33 ft.

Ice cover

Data on extent, thickness, and duration of ice cover on Lake Michigan have been compiled by the NOAA since 1973. During the winter season (December through March) the percentage of days with ice coverage in the Burns Harbor vicinity varied widely from 1973 through 1989, from 0 to 66 percent; the average is 34 percent. Ice thickness ranged from 1 to 20 in. Of the 32 storm events described above, 4 coincided with ice coverage ranging from 2 to 6 in. thick.

Damage and Maintenance History

The documented history reveals a harbor viewed as a "problem" by port users from an operations perspective, and by NCC from a maintenance perspective, since shortly after construction. A series of letters beginning in 1973

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from the Indiana Port Commission repeat complaints of excessive wave action and perceptions of breakwater damage. In their nomination to study Burns Harbor under the MCCP Program, NCC raised a suspicion that the design could be inherently deficient (see Volume II, Appendix 1C). This suspicion may have been related to an earlier internal memo, in which NCC cited four examples of failures (Burns Harbor excluded) occurring with two-layer, randomly placed armor structures in the Great Lakes.

There have been several serious instances of interior damage from wave action: barges have broken their moorings and been damaged, two vessels and two barges have sunk while moored at the Cargill grain dock, and north-facing revetments require frequent repair. A summary of the major damage events is provided in letters from the Port Commission and the operator of the grain dock (see Volume II, Appendix 1B). Repairs to the breakwater itself have been much more frequent and costly than anticipated.

The earliest damage to the structure is not well documented, but several documents make reference to an event during construction that resulted in damage, necessitating repairs the following year. One of the 32 maximum storm events (see Volume II) produced 12.1-ft (hindcast) waves on 15 December 1968 and could have been responsible for that damage. In an NCC memorandum dated 12 March 1970, the Burns Harbor breakwater was described as having no damage apparent from shore. The earliest well-documented damage/problems at the harbor are described in a letter dated March 9, 1973, from the Indiana Port Commission (IPC) to the NCC District Engineer. Observations after two severe storms (14.1-ft waves/29 January; 12.1-ft waves/15 February, 1973) revealed "some" stones lost, and "small gaps" as low as 3 ft below the design crest. Without additional elucidation, the letter states "... there was indication that there has been some subsidence of the breakwater."

There is no mention of increased wave transmission in this first letter; in fact, the breakwater is described as "... a very effective barrier to wave action during observed storms." Barely a week later, the U.S. Army Corps of Engineers tug *Moore* was sunk at the south end of the western arm inside the harbor during a severe storm. The hindcast significant wave height on 18 March 1973 was 12.8 ft with a 12-sec period.

Afterwards, IPC complaints of damage within the harbor and deterioration of the breakwater escalate. Damage is corroborated by a CERC field trip in October 1974, citing "extensive damage" and reduced freeboard, and by an NCC field trip in March 1975, which noted "...numerous gaps extending to within 2 to 4 ft of the lake level." The increasing evidence of structure damage culminated in a request from the NCC Operations Division to the Engineering Division in January 1975 for an investigation.

The requested investigation is described in an unpublished report, "Burns Harbor Indiana - Hydraulic Analysis for Performance of Federal Breakwaters for Period 1967 to 1975." When the first condition surveys, conducted in
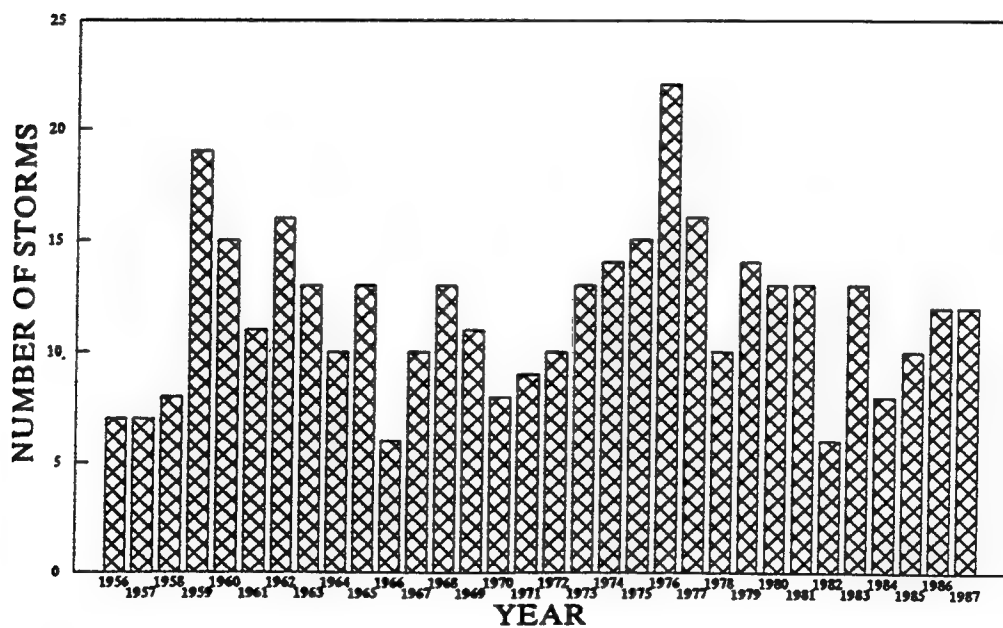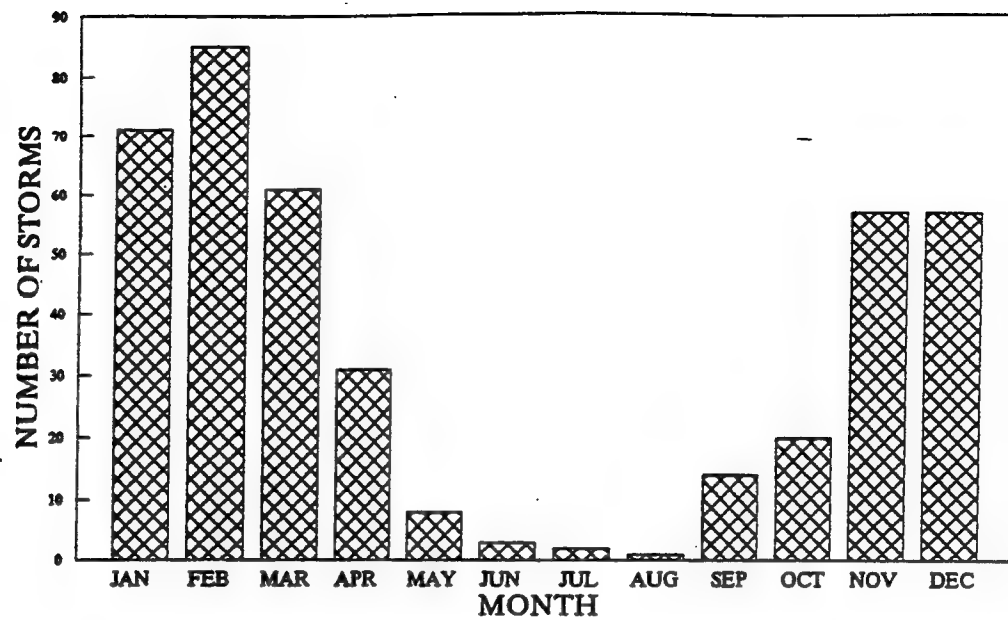
Figure 12. Distribution of hindcast storms from 1956 to 1987
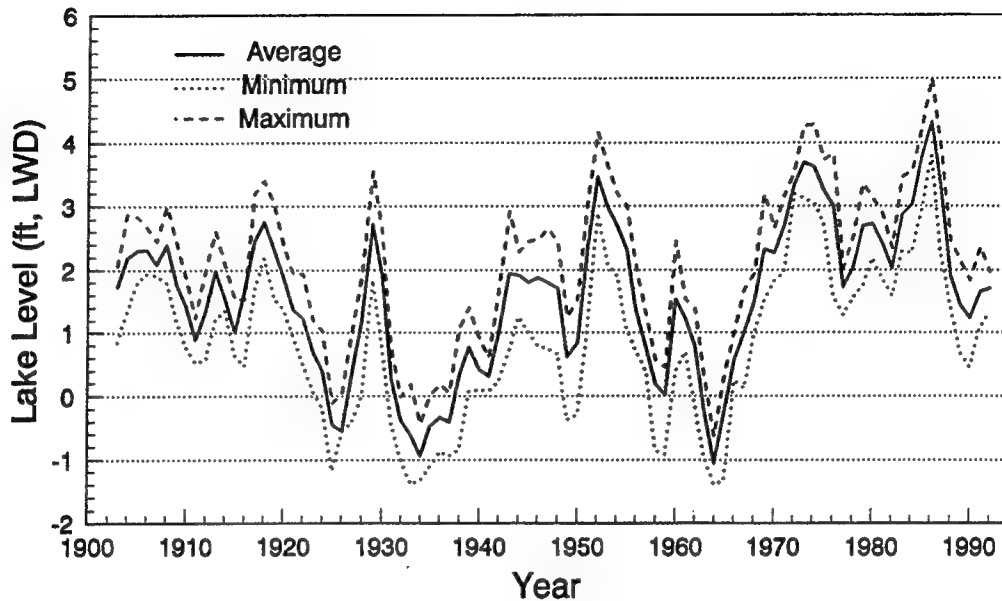
Figure 13. Historical Lake Michigan water levels from Calumet Harbor gage

April 1975, were compared to the as-builts, the problems associated with quantifying structure volume became apparent. Delineating changes, even qualitatively, is extremely problematic for a rubble-mound structure, particularly below the waterline. While there was visible damage to the armor layer, and damaged areas were calculable from the surveys, analysis indicated a net gain in area for the lakeside armor, and a substantial net loss in harborside armor. Survey error was postulated to explain the improbable growth in the lakeside armor (see "Lessons Learned" in Chapter 6 for additional discussion of survey problems).

Loss of harborside armor was attributed to survey error, inadequate armor size, damage from overtopping and/or transmission, or settlement. Other pertinent results were that section width at the LWD had reduced an average of 10 ft, or 18 percent, for the north breakwater, and 6 ft for the west breakwater, and the average elevation along the north breakwater crest was unchanged from the +14.0-ft design elevation. The report concluded: (a) the structure was exposed to 18-ft waves since construction, (b) the performance of the lakeside verified WES model results, which showed that the structure is stable for these waves, and (c) the significant damage on the harborside was not predicted.

Repairs of armor stone on both the lakeside and harborside were scheduled for the next 5 years, commencing in the summer of 1975. A report of inspection by the NCC District Geologist in July of 1975 declared the structure to be in better than expected condition, but the following summer, the IPC was repeating its request for repair work. A pattern of damage and repair continued through the 80's, with some sections receiving repeated maintenance. Figure 14 records the history of repairs by year, tonnage, and location. Amounts of repair stone placed each year and cumulatively are provided in Table 2; the total is 78 percent of the original armor amount. The maintenance stone placed on the breakwater, except for the 1989 repair, was limestone with a specific weight of 145 pcf. The 1989 repair utilized quartzite with a specific weight of 175 pcf. Figure 15 is a time line providing an overview of wave conditions (from WIS) and major events in the history of the structure.

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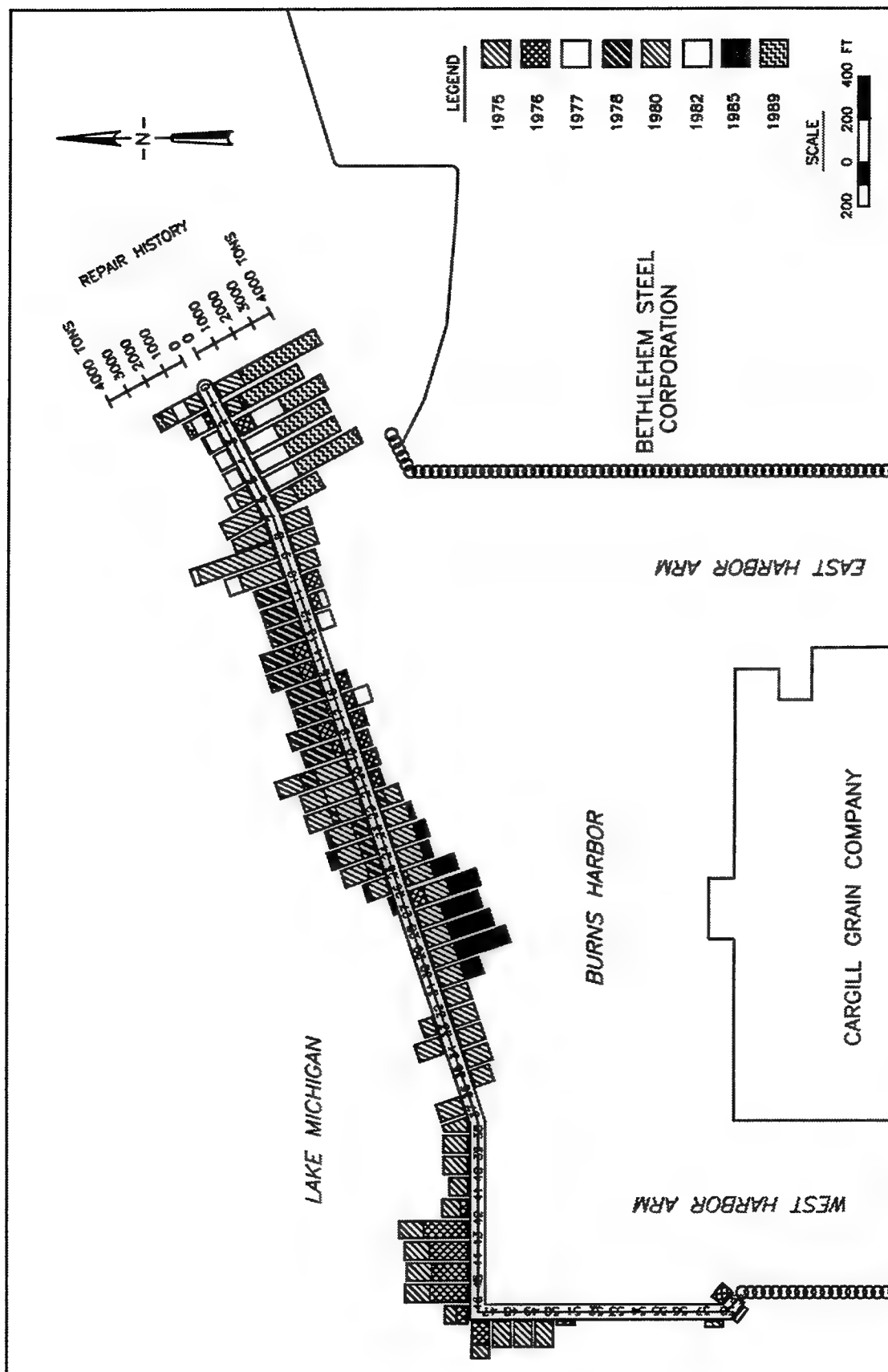
Figure 14. Breakwater repair history

Table 2
Annual and Cumulative Repair Stone Placed, Tons

Year	Harborside	Lakeside	Total
1975	2,028	14,703	16,731
1976	6,463	10,555	17,018
1977	1,373	8,904	10,277
1978	0	14,345	14,345
1980	20,944	26,385	47,329
1982	6,957	0	6,957
1985	11,083	750	11,833
1989	19,477	1,150	20,627
Total	68,325	76,792	145,117
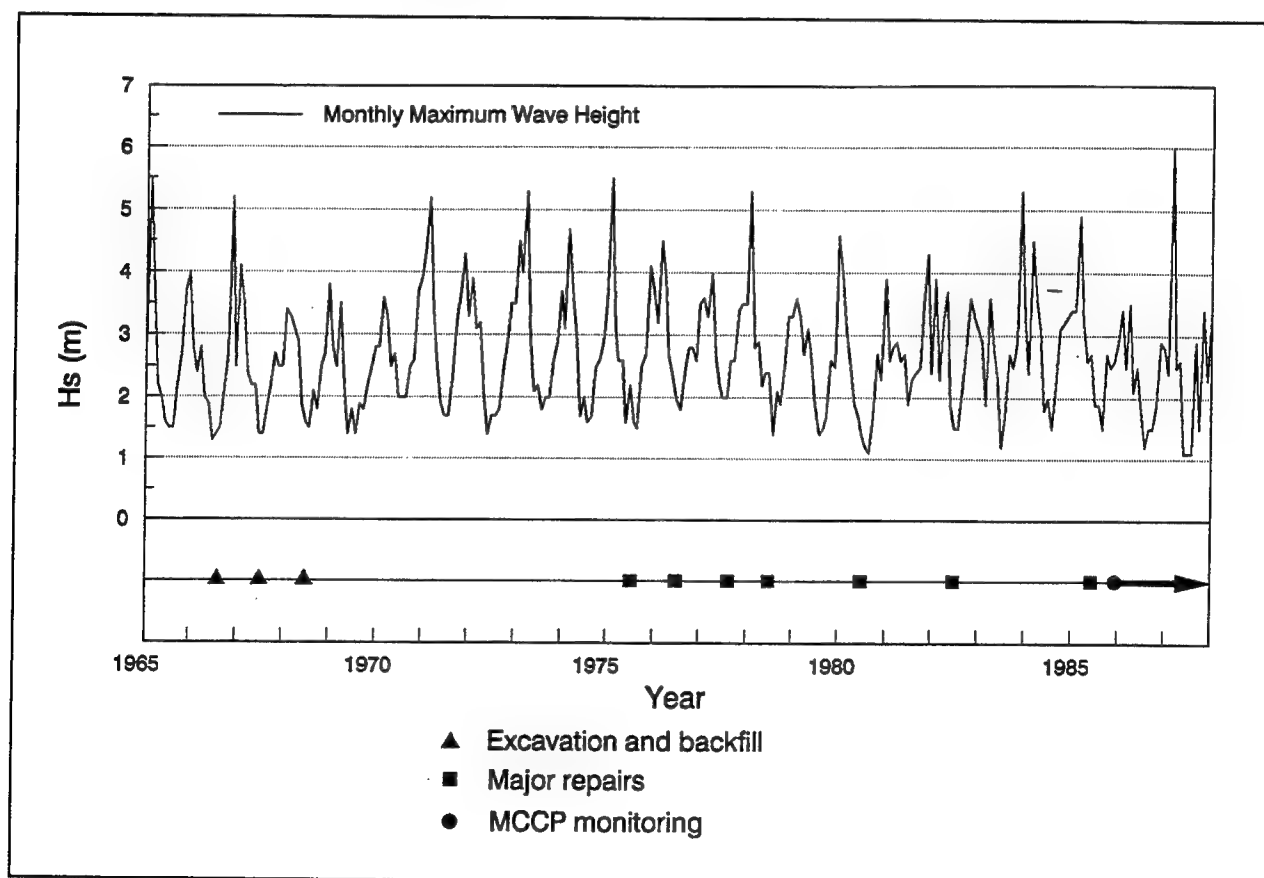
Figure 15. Time line of significant events with wave conditions

4 Monitoring Plan

Plan Development

The 1984 nomination of Burns Harbor for inclusion in the MCCP stressed the continuing need to maintain the crest elevation of the breakwater as the principal problem with the structure. The assumption at that time was that the loss of elevation was associated with foundation failure, inadequate armor stone stability, or both. Wave conditions inside the harbor were described as causing inconvenience to operations and damage to vessels. The original monitoring plan focused on collecting information on the following three technical areas: (a) structural stability, (b) geotechnical stability, and (c) waves and water levels. As the study progressed, the interaction of these three areas became apparent.

In addition to the three technical areas, the monitoring study was divided into three major subtasks. Those subtasks were historical review (HR), data collection (DC), and data analyses and synthesis (DA). HR involved collecting, indexing, and analyzing all data and records obtained before commencement of the study. This subtask was principally performed by NCC. DC refers to acquiring new information (i.e., monitoring the project), and includes data collected and paid for by MCCP funds, and incidental data acquired from other sources during the course of the study. CERC had principal supervision of this subtask, though NCC provided significant input. The final subtask, DA, was divided between NCC, CERC, and the Geotechnical Laboratory (GL) of WES, as well as outside consultants. Most of the data collected required reduction and analysis to be usable. The wave data analysis procedures, for example, reduce pressure measurements to wave parameters. Synthesis involved manipulation of either historical or new data or some synthesis of both to address a question.

Table 3 was developed to link each MCCP objective, in the form of specific questions of interest, with the elements of the monitoring plan (as denoted by technical area and subtask) that were expected to provide answers. The answer to the last six "bottom line" questions in Table 3 utilized results from many monitoring elements. Those six answers are of particular interest to the NCC for planning future action.

Table 3
Monitoring Plan Objectives and Elements

Program Objectives	Monitoring Plan Elements
1. Assess Project Attainment	
* Has the project met functional requirements re: harbor operations? navigation?	HR DC - waves DA - waves
2. Evaluate Design Procedures	
* Was the selection of the design wave appropriate for: overtopping? armor stability?	DC - waves DA - waves
* Was the design water level appropriate?	DC - water levels DA - water levels
* Was deposition adequately predicted?	HR
* Did the 3-D model correctly predict waves/ship response at: harbor entrance? berths?	HR
* Did the 3-D model correctly predict seiching in the harbor?	DC - waves DA - waves
* Did the 2-D model correctly predict: wave transmission? stone stability?	DC - waves DA - waves DA - waves, structure
* Did the foundation perform as predicted?	DA - structure, geotech
* Were the effects of ice predicted on: the wave conditions? the structure?	HR DA - structure
* Did the armor units endure weathering as expected?	HR
3. Evaluate Construction Methods	
* Was the structure constructed as designed re: foundation? bedding stone? core? armor layer?	HR DC - structural, geotech DA - geotech
* Were construction practices appropriate?	DC - structure, geotech
4. Evaluate O&M Methods	
* Has maintenance of the project been greater than predicted?	HR
* Have O&M practices been effective at: repairing damage? reducing damage? reducing transmission?	HR DC - waves, structure DA - structure

(Continued)

Table 3 (Concluded)	
Program Objectives	**Monitoring Plan Elements**
5. Solve Specific Problems	
* Has the breakwater experienced a failure? * How is wave energy entering the harbor? * How can wave energy in the harbor be reduced? * What is the reason for the frequent maintenance? * How can maintenance costs be reduced? * What is the prognosis for the present structure?	All elements utilized

The remainder of this chapter describes the data collection and analysis phases for the three technical areas, including the approach used in obtaining and reducing the data or in performing the analyses. Results from each element are summarized in Chapter 5 (historical review is described in Chapter 3). Discussion of the results (Chapter 6) will be provided, as far as practical, in the form of answers to the questions in Table 3.

Data Collection

Structural data collection

Site inspections. Burns Harbor is within a 2-hr drive from the NCC offices, making periodic site inspections practical. Inspections were typically documented by photographs and were conducted on foot, from boats, and from aircraft under various conditions. The purposes of inspections were as follows: (a) to assess the condition of the subaerial portion of the structure, in particular the effects of weathering on the armor units; (b) to document the condition with photographs; (c) to observe and videotape the response of moored vessels during energetic wave conditions; and (d) to visually observe the nature and patterns of wave activity in the harbor. Photographs were grouped chronologically and spatially, by section, to ascertain trends and patterns.

Dive inspections. Though underwater visibility in the vicinity of the breakwater was limited, diver inspections conducted mostly by touch still offered the only way of assessing the condition of the structure in any detail below the water surface. Hands-on inspection provided the best alternative to obtain detailed information on: the submarine armor condition, such as placement/ orientation, extent of coverage, and location of voids; extent and condition of other layers, such as W/10 stone and bedding; and identification of features observed with acoustic surveys.

A total of four dive inspections were conducted of both the lake- and harborsides of the structure by dive teams from CERC and/or the U.S. Army Engineer District, Detroit (NCE). In addition to reporting their observations, the divers collected sediment samples and probed below the mudline near the structure toe. Diver's reports are reproduced in Appendix B of this volume.

Cross-section surveys Cross-section surveys of the entire structure were conducted in 1967 (as-built) and in 1975. Partial surveys were conducted in 1985, 1989, and 1992 (western arm only). The 1967 cross-section survey results were plotted and overlain with results from subsequent surveys (see Volume II, Appendix 5A) to illustrate historical changes in each cross section from 1967 to 1992. While the MCCP study did not support these bathymetric surveys, analysis of the available survey data was an important element of the study.

There is no record of the technique used to obtain the as-built surveys. The cross sections follow the design profile too closely to represent discrete elevations at normal spacing on a structure with such a varied profile (see Figure 11). Spacing of recorded soundings varied from 10 to 40 ft. It is likely some form of smoothing was utilized on the data.

The 1975 survey was performed by the Kewaunee field office, NCC. Rod and level were used for the subaerial portion at 100-ft stations; a Bludworth model ES130A acoustic echosounder obtained sections for the submerged portion at 25-ft stations. Elevations were spaced every 5 ft both above and below water.

The 1985 survey was conducted by the Grand Haven Area Office of the NCE. Elevations at 5-ft spacings were obtained at 100-ft intervals from stations 0+00 to 15+00, and from stations 23+00 to 34+00. Subaerial portions were surveyed with a rod and level; submerged portions were measured with a sounding basket lowered from the survey vessel.

The Saginaw Area Office, NCE, conducted the 1989 survey with sections every 50 ft from stations 7+00 to 58+00. The approach was identical to the 1975 survey, including use of the same echosounder. The technique was changed, however, in that local extremes in elevation (peaks and voids) were deliberately avoided in an attempt to smooth out the profile. Some portions of the profile near the waterline were also omitted due to slippery, dangerous footing. Later in 1989, as-built sections were obtained after completion of repairs from stations 0+00 to 6+50 by the construction contractor. The western arm was re-surveyed in 1992 with rod and level above, and sounding basket below water by a private contractor. The last two surveys made no attempt to smooth the profiles.

Side-Scan-Sonar (SSS) Surveys. Four SSS surveys were conducted during the course of the MCCP study. SSS surveys provide an overhead view of a swath of the lakebed beneath a towfish pulled through the water column. Differences in bottom composition or slope are revealed as different shades on the

plot. Surveys were made with two brands of sonar: a Klein model 531T operating at 500 kHz on 20 - 21 August 1986; and an EG&G Model 260 on 25 September 1985 (at 500 kHz), on 23 September 1987 (at 100 kHz), and on 23 September 1992 (at 500 kHz). The towfish was suspended approximately 3 m below the water surface in each case. Survey lines were run parallel to the breakwater at various distances from the structure and at various scales to reveal both large and small-scale features. The purposes of the SSS surveys were to: (a) image the features of the structure and surrounding lakebed; (b) assess the condition of the structure toe; (c) determine the extent of core-stone projecting beyond the toe of the structure; and (d) ascertain the presence of displaced armor units on the lakebed.

Geotechnical data collection

Subbottom sonar (SBS) survey. Low-frequency acoustic energy from an SBS penetrates the bottom, revealing differences in composition of sediment layers. The SBS survey was conducted in conjunction with the 1986 SSS survey and utilized an ORE model 140 transceiver operating from a 3.5-kHz tuned transducer sound source. SBS lines were run perpendicular to the breakwater on the lakeside at stations 16+00, 25+00, and 33+00, and on the harborside at stations 6+00, 25+00, and 35+00 (later referred to as SBS lines 16L, 25L, 33L, 6H, 25H, and 35H, respectively). The purpose of the SBS was to: (a) identify the condition of the foundation at the toe, particularly to ascertain whether the backfilled sand layer overlaying the clay layer projected beyond the toe, as designed; and (b) determine if armor stone could be detected below the sediment, beyond the apparent toe of the breakwater.

Sediment samples. During the 1986 SSS survey, a grab bucket was used to obtain samples of the four distinctive sediment types apparent from the SSS sonogram - fine sand, silty clay, coarse sand, and cobbles.

Wave and water level data collection

Wave measurements. Wave gages were installed at locations 1 through 4 as shown on Figure 3. Site 1 is located directly in front of the breakwater and measures the combined incident plus reflected wave field. Site 2 is behind the breakwater at approximately the same station and measures the total wave energy transmitted into the harbor. It was situated to minimize influence by wave energy coming through the entrance. Site 3 is directly in front of the highly reflective grain dock that experienced damaging wave conditions. Site 4 was selected to measure incident waves approaching the breakwater. It is located to the west of the breakwater in front of a beach assumed to produce negligible reflection.

Ideally, all four gage sites would have been instrumented simultaneously, but the budget constrained the number of available gages. The gages were self-contained, single-point pressure sensors mounted on steel frames on the

lake bottom. The sampling scheme is constrained by battery and memory capacity (technical specifications of the gages can be found in Volume II, Appendix 2A). In order to obtain data over the winter storm season when retrieval is impractical, waves were sampled at 1 Hz for 1,024 sec every 3 hr.

Water level measurements. In addition to the lake levels from the gage at Calumet Harbor, mean water depth over the gage was obtained for each wave record from the bottom-mounted wave gages at Sites 1-4.

Data Analysis

Structural analysis

Details of the structural analysis are found in Volume II, Chapter 5.

Structural condition analysis. The current condition of the structure was evaluated through comparison of the available surveys to the design template (See Volume II, Chapter 5). All available cross sections for each 100-ft station were digitized and entered into AUTOCAD for calculation of areas, area changes between surveys, and various statistical parameters.

The rubble-mound portion of the breakwater from Station 0+00 to 57+00 was subdivided into eight segments, as follows:

a. Segment 1 = 0+00 to 6+00[1]

b. Segment 2 = 7+00 to 16+00

c. Segment 3 = 17+00 to 22+00

d. Segment 4 = 23+00 to 31+00

e. Segment 5 = 32+00 to 37+00

f. Segment 6 = 38+00 to 46+00

g. Segment 7 = 47+00 to 50+00

h. Segment 8 = 51+00 to 57+00

Partitioning the structure into segments allowed some reduction, through averaging, of the uncertainty inherent in surveys of individual cross sections, while retaining the capability to localize variations in the condition of the structure. The layout of the eight segments is shown in Figure 16. The segments were defined so each represents a zone of constant orientation where relatively equal amounts of maintenance stone were placed on the harborside and/or lakeside locations.

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[1] In calculations involving cumulative properties, the segments were extended to eliminate gaps, as shown in Figure 16.
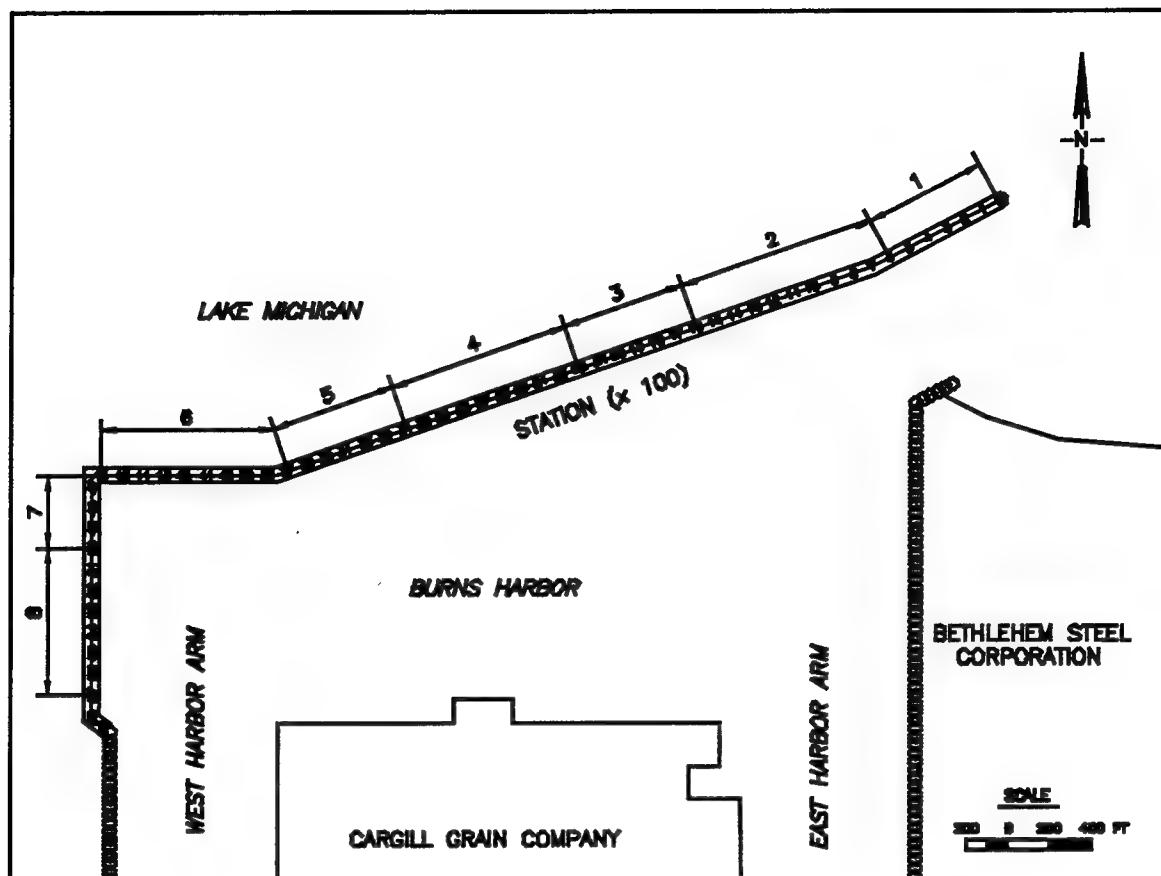
Figure 16. Breakwater segments

Since the horizontal control utilized over the years varied, it was not possible to accurately delineate a constant structure center line between subsequent surveys. As a result, temporal changes in area were not attributed to either the lakeside or harborside of the structure. However, since vertical control was maintained adequately between surveys, each cross section was divided into an upper and lower region, as illustrated in Figure 17. The intent was to investigate differences between the upper region, which is more likely to lose stones by wave energy and/or ice action, and the lower region, which is more likely to gain volume through relocation of those same stones. The upper region extends from the crest (+14 ft LWD) to approximately one design wave height below the design still-water level (-10 ft LWD). The lower region begins at -10 ft LWD and extends to -30 ft LWD. Though the structure extends below -30 ft at many stations (as far as -41 ft on the lakeside), a single, common lower boundary was selected for all sections, and the remainder of the structure was ignored. The design template is 1,272 ft^2 for the upper region and 2,380 ft^2 for the lower region.
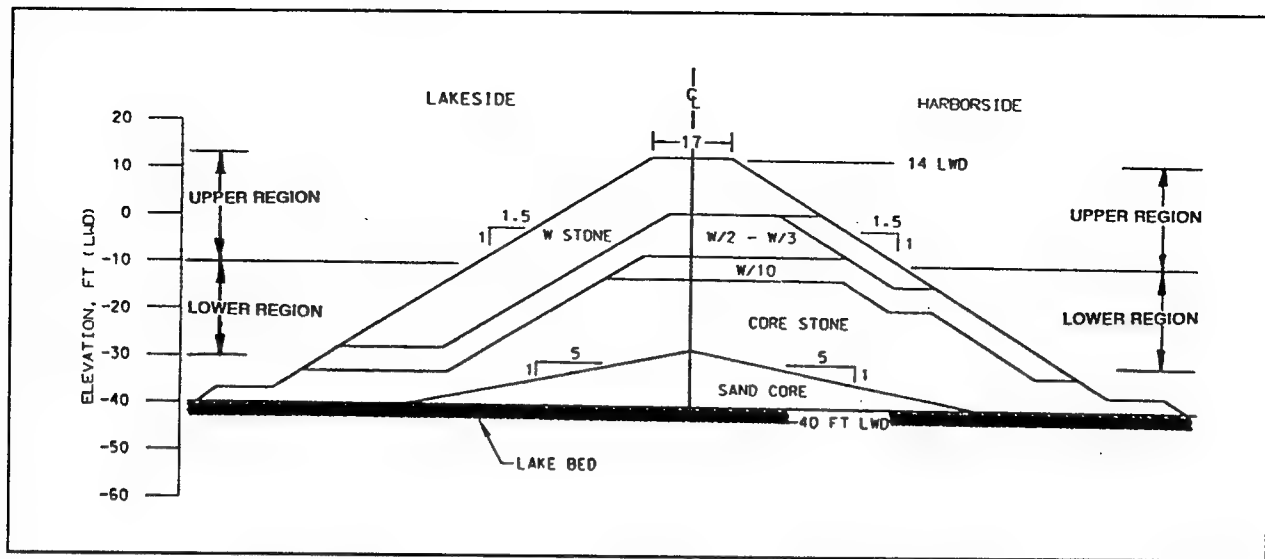
Figure 17. Vertical subdivision of cross section

The principal statistical parameters used for comparison are the mean cross-sectional area $\bar{x}$ and the variance of the cross-sectional areas s^2 for each breakwater segment. Minimum and maximum cross sections for each segment were also retained. The mean is compared to the design template area, and the variance is used as a relative evaluation of the irregularity of the segment. Segments that showed mean areas at or above the template, with small variance, are considered either undamaged or adequately repaired. Segments with deficient areas and small variances are assumed to have settled. Large variances are considered indicative of slope instability or nonuniform repair.

Comparisons and rankings were made to examine temporal changes of these statistics for each segment from 1967 to 1989 ("within-segment comparisons"); and differences between segments at the time of each survey ("across-segment comparisons"). This procedure provides both a picture of the evolution of the structure, and a relative ranking of the segments to each other. Validity of the calculated changes in these parameters for making inferences about the actual structure was tested using the Smith-Satterwait procedure for evaluating statistical significance.

Structural stability analysis. The approach used in the structural stability analysis by NCC addressed whether existing methods for estimating break-water damage correctly assessed the performance of Burns Harbor breakwater (Volume II, Chapter 5). Deterministic investigation of wave damages focuses on the stability of individual armor units (stones) under wave attack. Rubble-mound breakwaters are classified as statically stable structures. This implies that very little or no movement of the armor units is acceptable. Any displacement of stones is referred to as damage. Displacement of armor units to the toe of the breakwater (below -30-ft LWD) is considered damage since it effectively removes the armor units from the functioning portion of the breakwater.

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For the Burns Harbor breakwater, two modes of damage were examined: settlement and wave damage. Analyses were conducted to estimate the changes (damages) in the breakwater using deterministic relationships prescribed by theory. The cross-sectional analysis using three dates of breakwater surveys provides a method of measuring actual changes along the breakwater accounting for survey error. Actual changes in the breakwater cross section were compared to estimated changes for each breakwater segment.

Estimating damages using the average annual wave climate is one way of calculating expected damages. Another method is to use the actual wave climate experienced, and then correlate it to the Hudson relationship to estimate damages over the life of a structure. This was done for Burns Harbor breakwater using the WIS conducted by the CERC which documented specific storm events over the life of the Burns Harbor breakwater. Seven storm events exceeded the 15-ft design wave height. These seven events were used to estimate damages for the breakwater between 1967 and 1987. This estimate will be applicable for the time period since construction as opposed to the 50-year design life time period of the average annual wave climate estimate.

Volumetric analysis. Because the initial volumetric calculations used in the structural condition analysis did not extend below -30 ft, and thus did not answer whether unaccounted-for stone was, in fact, located at the toe of the structure, this approach could not ascertain whether significant settlement occurred. To better identify the damage mode, and thus the reason for the frequent maintenance, the AUTOCAD-based volumetric analysis performed by NCC was supplemented with hand calculations of area changes in the cross-section surveys below -30 ft, and extending out to the horizontal lakebed on either side of the structure. In addition, 100-ft "buffer" zones between segments, utilized in the stability analysis, were included in the overall volumetric analysis. Changes in area between the as-built and the latest available survey for each section were obtained with a planimeter. The change in area was converted to tons in the same manner as described in the structural stability section. A description of the hand calculations is provided in Appendix C of this volume. Net gain or loss in tonnage for the entire cross section above the lakebed for each segment could then be compared to maintenance stone added in each segment.

Geotechnical analysis

Settlement recalculation. The foundation was reanalyzed by the WES GL to compare the original results with those obtained using more recent analysis techniques, and to estimate additional settlement attributable to a hypothetical construction scenario that may have affected the foundation's performance (see Volume II, Chapter 4). A separate reanalysis was performed by NCC, which included the maintenance stone.

Wave and water level analysis

The purposes of the wave measurement and analysis effort were as follows: (a) evaluate the original selection of the design wave, (b) measure the breakwater's reflection and transmission characteristics, (c) evaluate the 2-D physical model transmission results, (d) monitor the actual wave conditions in the harbor, and (e) determine the mode by which wave energy entered the harbor. The purpose of the water level analysis was to evaluate the selection of the design water level.

Wave data reduction. Spectral analysis of the pressure time-series provides a one-dimensional energy spectrum of the water surface. In the following discussion, the terms wave height and period will refer to an energy-based wave height H_{m0} calculated from the zeroth moment of the one-dimensional energy spectrum (generally equivalent to significant wave height H_s), and the period T_p associated with the peak of the energy spectrum.

Wave extremal analysis. The purpose of the extremal analysis was to calculate the probability of occurrence of wave heights at the structure (the design wave for various return periods) for comparison to the original design wave selection. Improved results were expected since the analysis used measured wave data and numerical models not available when the project was designed. These results will also be useful for any future planning/design activities for projects in the vicinity of Burns Harbor.

Data collected at Burns Harbor were not intended to provide climatic information since the study was not of sufficient duration to collect a statistically significant record. Wave statistics were based primarily on hindcast wave data provided by the WIS. The hindcast was validated using measured data from Site 4, calculated incident waves from Site 1A, as well as measurements from a deepwater wave gage in Lake Michigan (Station 45007) operated by the National Data Buoy Center (NDBC) since 1981 (see Figure 1). To best utilize the available wave data, the hindcast was supplemented with these measurements in an integrated extremal analysis (Volume II, Chapter 3). Conservative results were assured by selecting the higher of the measured or the hindcast data.

Wave reflection/transmission. The breakwater's wave reflection characteristics were obtained by analyzing simultaneous measurements from Sites 1 and 4. To describe the relationship between measurements at these two sites, an energy-based method was used to determine a reflection coefficient K_R, as a function of incident wave height. The transmission function of the breakwater K_T was similarly calculated from simultaneous measurements at Sites 1 and 2. Since the measurements at Site 1 contained both incident and reflected energy, they were transformed by the reflection function K_R before calculation of K_T. Calculated incident wave data from Site 1 are referred to as from "Site" 1A. Finally, the measured transmission characteristics were compared graphically to the physical model results.

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Harbor wave conditions. Wave conditions in the harbor were obtained simultaneously at Sites 2 and 3. The purpose of the gage at Site 3 was to document the amplification of the wave conditions in the harbor by the reflective, vertical sheet-pile face of the grain dock. Comparison of the incident (that is, from Site 2) and reflected waves provided a measure of the reflection coefficient of the dock face.

Wave energy can enter the harbor directly through the entrance, by transmission through the breakwater, and by transmission over the breakwater (overtopping). Total energy transmittance can be due to all three modes, either separately or in combination, depending upon the direction and intensity of the incoming waves. The dominant mode during various conditions was determined by analysis of the incident and transmitted spectra, supplemented by site inspections and aerial photographs.

Harbor seiching. Low-frequency seiching had not been identified as a problem at Burns Harbor. The 3-D model study used linear, shallow-water wave theory to predict that the dominant modes of oscillation would be 60 sec in the west harbor arm, 50 sec in the east harbor arm, and 50 sec in the turning basin in the east-west direction. Little, if any, incident wave energy to excite these modes is anticipated in Lake Michigan. With self-recording gages, compromises must be made between sample size and available power and memory in the gage. The sampling scheme of the gages (1,024 one-Hz samples every 3 hr) was selected to optimize deployment times while measuring wind-generated gravity waves (i.e., frequencies from 0.03 to 0.30 Hz). Although spectral analyses of the time series was extended into lower frequencies (as low as 0.0078 Hz, or 128-sec period), confidence in measured energy levels at frequencies less than 0.03 Hz, where seiche energy dominates, is not high because the time series of 1,024 sec provides few samples of these longer waves.

Water levels and storm surge. The century-scale length of record from the Calumet water level gage permits a reliable calculation of frequency of occurrence of water levels. Mean water levels were obtained from the wave gages by averaging the pressures for every wave record and using hydrostatic conversion to depth. Though these gages were not surveyed into any datum, they do provide a record of relative water level fluctuations over the deployment. If two or more gages are located within a limited area with no net flow occurring over the deployment interval, the long-term mean can be assumed to represent a constant datum for all the gages (McGehee, McKinney, and Dickey 1989). Though the conversion was not performed in this study, that assumption permits MCCP-measured water levels, which provide detailed measurement of the storm surge directly in front of the breakwater, to be converted to the same datum as the Calumet gage.

5 Monitoring Results

Data Collection

Structural data collection

Site inspections. Inspections provided valuable qualitative information and insight into the processes dominating the system. Photographs of the subaerial structure were used in identifying damage patterns and documenting the general condition of the structure. Aerial photographs were helpful in illustrating wave transmission through the breakwater, as opposed to through the entrance, as the principal mode for wave energy to enter the harbor, even under moderate wave conditions. Ground-level photographs identified isolated instances of armor units that had shifted and/or fractured in place. Videos of storm conditions in the harbor provided estimates of the frequency response of moored barges. They verified that barge heave, pitch, and yaw motions were in the 10- to 12-sec periods of the incident wave energy.

Dive inspections. Reports of diver surveys for the harborside in July 1988 and harbor and lakeside in July 1992 are contained in Appendix B. The divers reported that in most locations, armor stone extended all the way down the section to the toe of the structure, on both the lake and harborsides. Reports differ on the configuration of armor stone at the lakebed, with some inspections reporting stone beyond the toe, and others describing a well-defined toe. Both the lakeside and harborside toes were visible down to the lakebed in the 1988 survey; the harborside toe was exposed down to the sand lakebed at -40 ft in the same areas covered by mud to -20 ft in the July 1992 survey. The typical profile below the water was more random than the visible portion of the structure. The distribution of corestone projecting out from the toe was irregular; many sections had no visible corestone, so armor rested directly on the lakebed. Around Station 15+00, the corestone began at -30 ft, and projected lakeward over 50 ft horizontally to -40 ft (this thick layer of corestone is also shown on the as-built surveys in the vicinity of Station 15+00). In the 1992 inspection the armor was not covered by sediment on the lakeside, but mud appeared to cover much of the toe on the harborside. The thickness of this mud cover increased toward the western end, where the interface between the armor and the mud occurs as shallow as -20 ft.

Side-scan sonar surveys. SSS is a useful tool for determining the general plan view of a rubble structure and surrounding lakebed, in particular the interface between the toe and natural bottom. It is less effective in imaging the condition of the slope of the structure itself. Figure 18, a sonogram of the lakeside from the 1985 survey, is typical of images used to assess the condition of the submarine portion of the structure. Labelled features on Figure 18 were identified from sediment samples and verified in subsequent dive inspections. Figure 19 is a sonogram of the lakeside between Stations 35+00 and 45+00 taken at closer range.

There is no indication of a significant number of armor units projecting beyond the visible toe of the structure, which is relatively straight along the structure's length. There is no indication of a major slide-type failure resulting in armor units spilling onto the lakebed at any section. Although several individual units can be detected as far as 50 ft from the toe between Stations 35+00 and 45+00 (Figure 19), there has been no change in their amount or location in subsequent surveys, and there is no way to ascertain whether their presence is due to construction/repair activity or wave action. Within the limited resolution of the sonogram, no discernible change occurred to the structure toe from 1985 to 1992, implying that any redistribution of armor stone during that time interval was confined to the slope of the structure.

The distribution of the corestone is clearly visible on the lakeside SSS sonograms (Figure 18). The plan called for an apron of corestone to project outward 15 ft from the next higher layer. Generally, corestone projects out from the eastern half, in places more than 50 ft, while it is rarely visible on the western half. The uneven pattern raises questions about the extent and thickness of the core layer under the structure, but no practical means were available to determine its condition.

Also visible in Figure 18 are the remnant mounds of consolidated clay excavated for the foundation. The mounds are in a linear form about 100 to 150 ft from the structure toe (about the swing distance of a large crane). Most of the unconsolidated excavated clay has likely been removed by wave action over the years, but significant amounts remain. Some of the piles project as high as 10 ft above the lakebed.

Four discrete areas of "coarse sand" are easily delineated by the presence of sand ripples. The ripples are on the order of feet in length and inches in height. Diver inspection verified the sand piles extend up to and over the lower portions of the clay mounds. This arrangement indicates the coarse sand was placed after the clay was deposited.

SSS sonograms of the harborside reveal a featureless harbor bottom and a relatively straight toe. Figure 20 is a typical portion from Station 30+00 to 40+00 at a larger scale than used for Figure 18 of the lakeside. There is no indication of corestone at any place on the harborside. There is no indication

of a significant sliding failure or distribution of any armor units on the lakebed.

The SSS revealed large curving tracks on the harbor bottom at the entrance to the harbor (Figure 21). These features could be the result of propeller wash from deeper draft vessels entering the harbor. They are also consistent with descriptions of a technique used by larger vessels to negotiate the sharp port-side turn required to approach the Bethlehem Steel mill. While underway westward in the entrance, a port anchor is dropped on a short scope and used as a pivot point to assist in the turn. The curved tracks are likely dramatic evidence of that method's use.

Cross-section surveys. All surveys were digitized and analyzed using AUTOCAD to allow intercomparisons and volumetric computation (Figure 11). These values were used in the cross section and stability analyses below.

Geotechnical data collection

Subbottom sonar surveys. Track lines 16L, 25L, and 33L are shown on Figure 18 with hash marks labelled at corresponding fix marks (A, B, C,...) on the SBS sonograms to identify the features. Figure 22 is the sonogram of line 25L. The shallow relief of the coarse sand deposit, the abrupt outline of the clay mound, and the beginning of the breakwater slope are easily identifiable. Some layering is discernible in the sediments away from the structure.

Near the structure toe, where the bottom consists of sand backfill and/or corestone, the SBS signal deteriorates. The acoustic pulse must exceed a higher energy threshold to penetrate the relatively reflective sand, compared to mud or clay. As the source output level is increased, ringing of the acoustic pulse (multiple echoes due to repeated bouncing between the surface and the bottom, and from sidelobes reflecting from the highly reflective armor) masks the return signals from just below the bottom. As a result, there is insufficient detail in the image close to the structure to determine whether the foundation was constructed as designed on the lakeside.

On the harborside, the SBS confirms the featureless bottom seen on the SSS sonogram. Figure 23 is the line 35H. The faint return from the soft clay as it slopes upward to about -20 ft is barely visible at the right end. Its shape is inconsistant with sedimentation due to material settling from above. There is no evidence of a distinct hard return as would be caused by armor stone below this slope. The curved reflection labeled "sidelobe" comes from strong reflections from the armor off to the side of the transducer as it approaches the structure.

In contrast, line 6H (Figure 24), which traverses the entrance channel, indicates a harder layer 7-10 ft below the lake bottom, or at -45 ft. Approaching

Figure 16 — SSS sonogram of breakwater (lakeside)

BEDDING STONE

SAMPLE 4

SAMPLE 2

WATER COLUMN

SBS LINE 25L

SBS LINE 33L

BEDDING STONE

BREAKWATER TOE

D

SAMPLE 1

C

SBS LINE 16'

WATER COLUMN

SAMPLE 3

3

BREAKWATER TOE

STATION 1 + 00
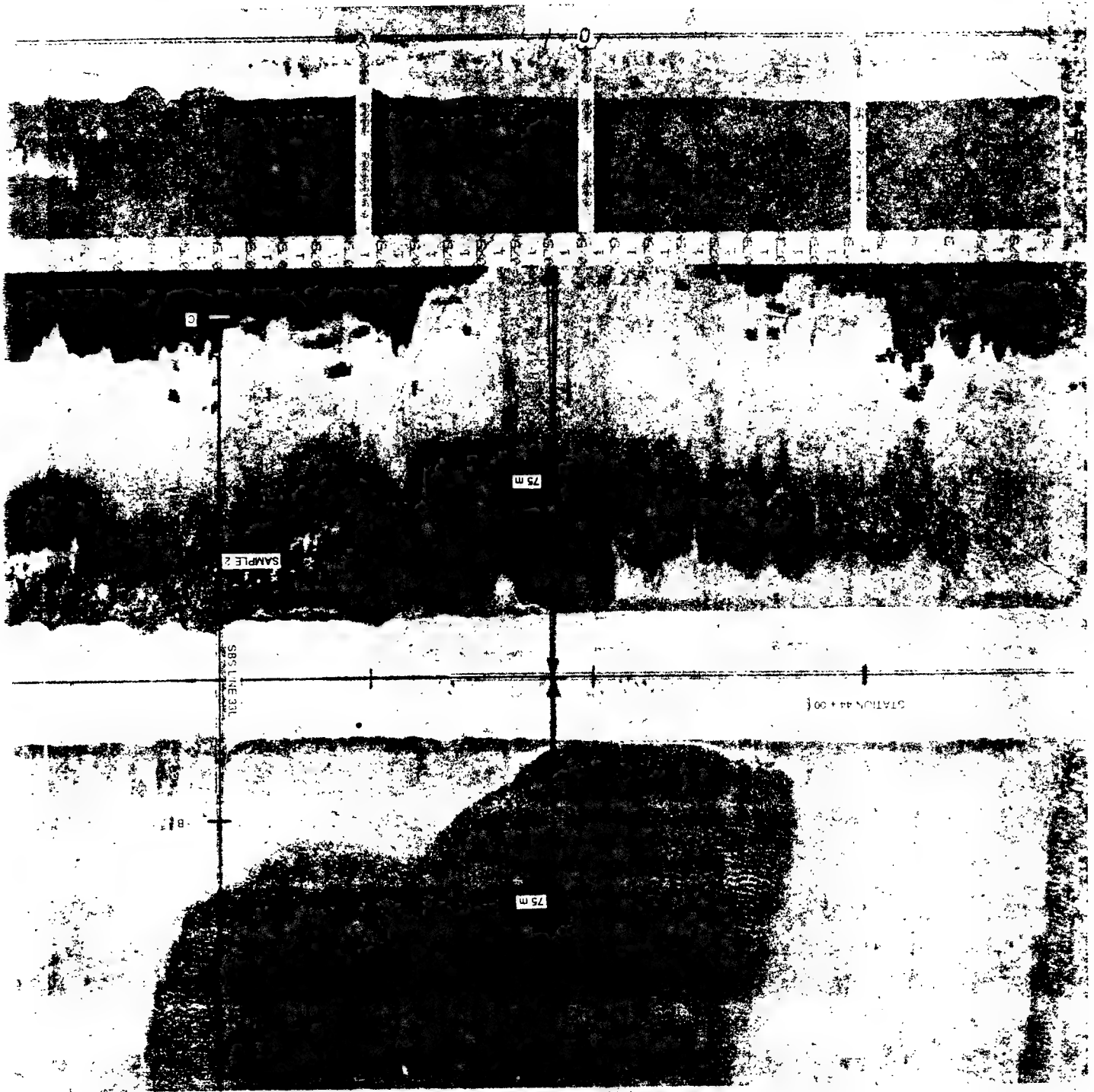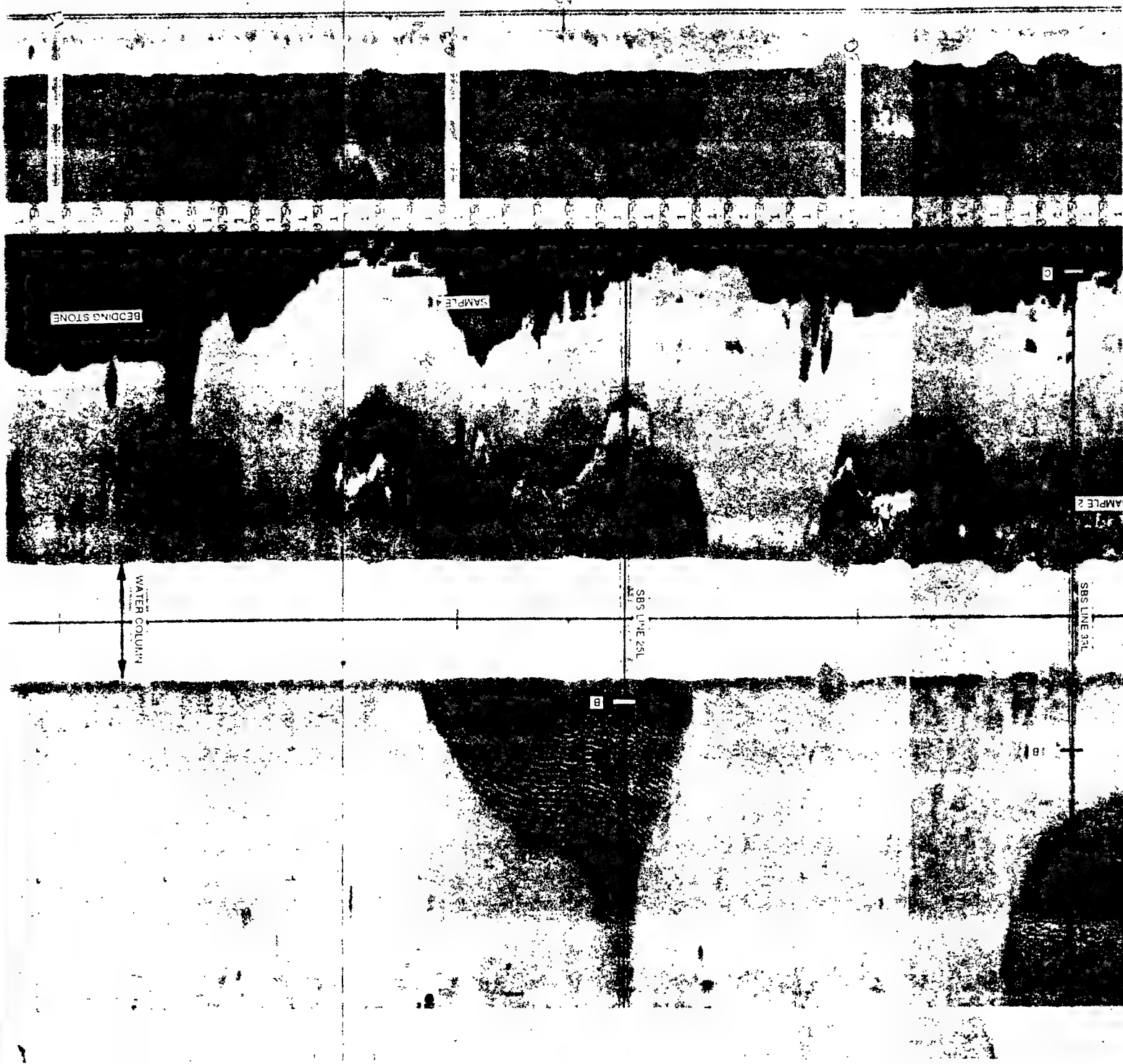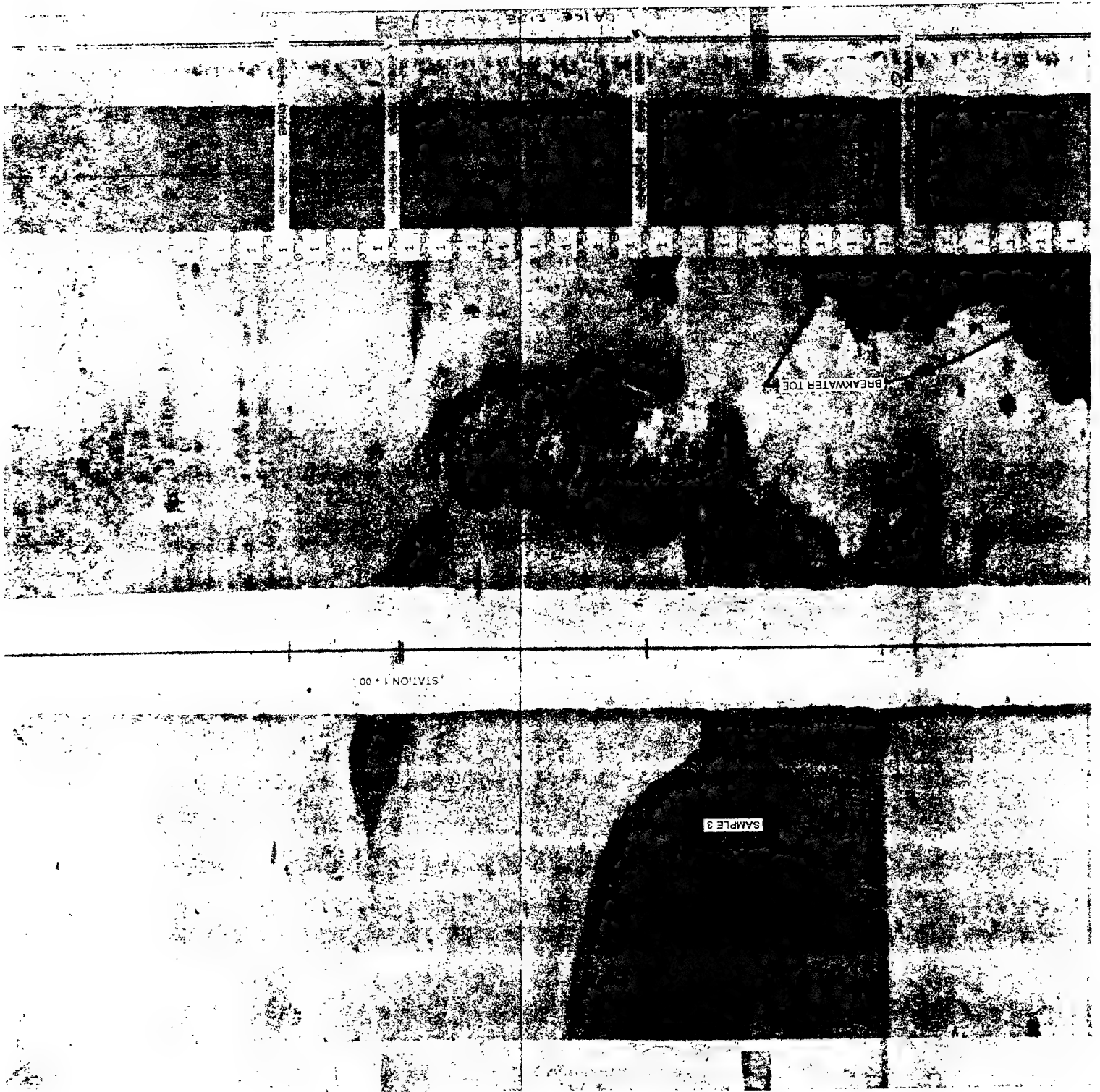
SAMPLE 3

④
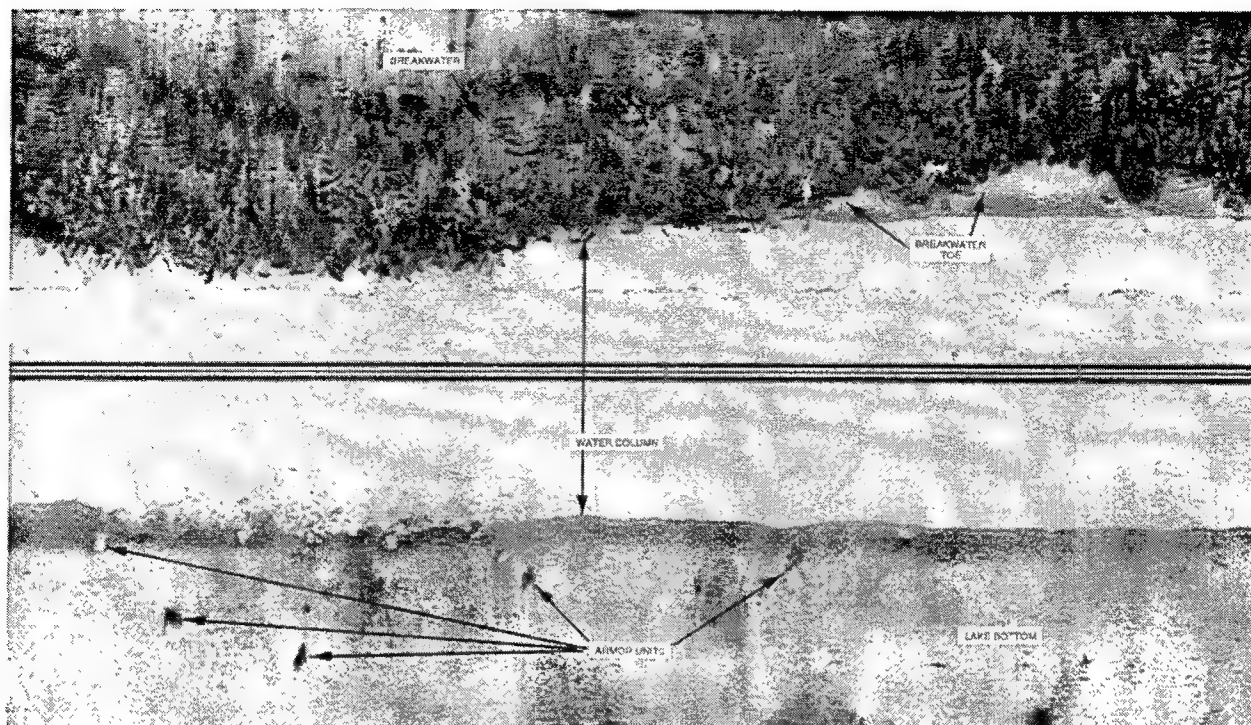
Figure 19. Lakeside SSS record showing individual armor units

the structure slope, some reflectors are visible below the upper mud layer, indicating stone projects further out than the visible toe at -32 ft.

Sediment samples. The approximate positions of the four samples are shown on the lakeside SSS sonogram (Figure 18). The samples were not analyzed following the sampling period, but were sufficiently distinct to allow visual identification. Sample 1 was a fine to medium sand indicative of the natural lakebed. Sample 2 was stiff clay: it was obtained from a mound and is believed to be remnant natural clay excavated from the site. Sample 3 was a coarse sand/gravel mixture. Sample 4 consisted of several fist-size cobbles which were not analyzed or measured.

Wave and water level data collection

Wave measurements. Data return was about 80 percent over the combined deployment intervals. Volume II, Appendix 2A contains plots of the reduced parameters H_{mo}, T_P, and depth for the entire data set. A height threshold of 0.2 m was applied to the reduced data, since estimates of H_{mo} and T_P for low energy conditions are questionable, and waves below 0.2 m have no engineering significance for this study. Figure 25 is an example of the plotted values for December 1987 at Site 4. As indicated above, no data are shown for the rather extensive intervals when the measured wave height was less than 0.2 m. Figure 26 is a typical sea surface energy spectrum from Site 1 on 8 February 1987.
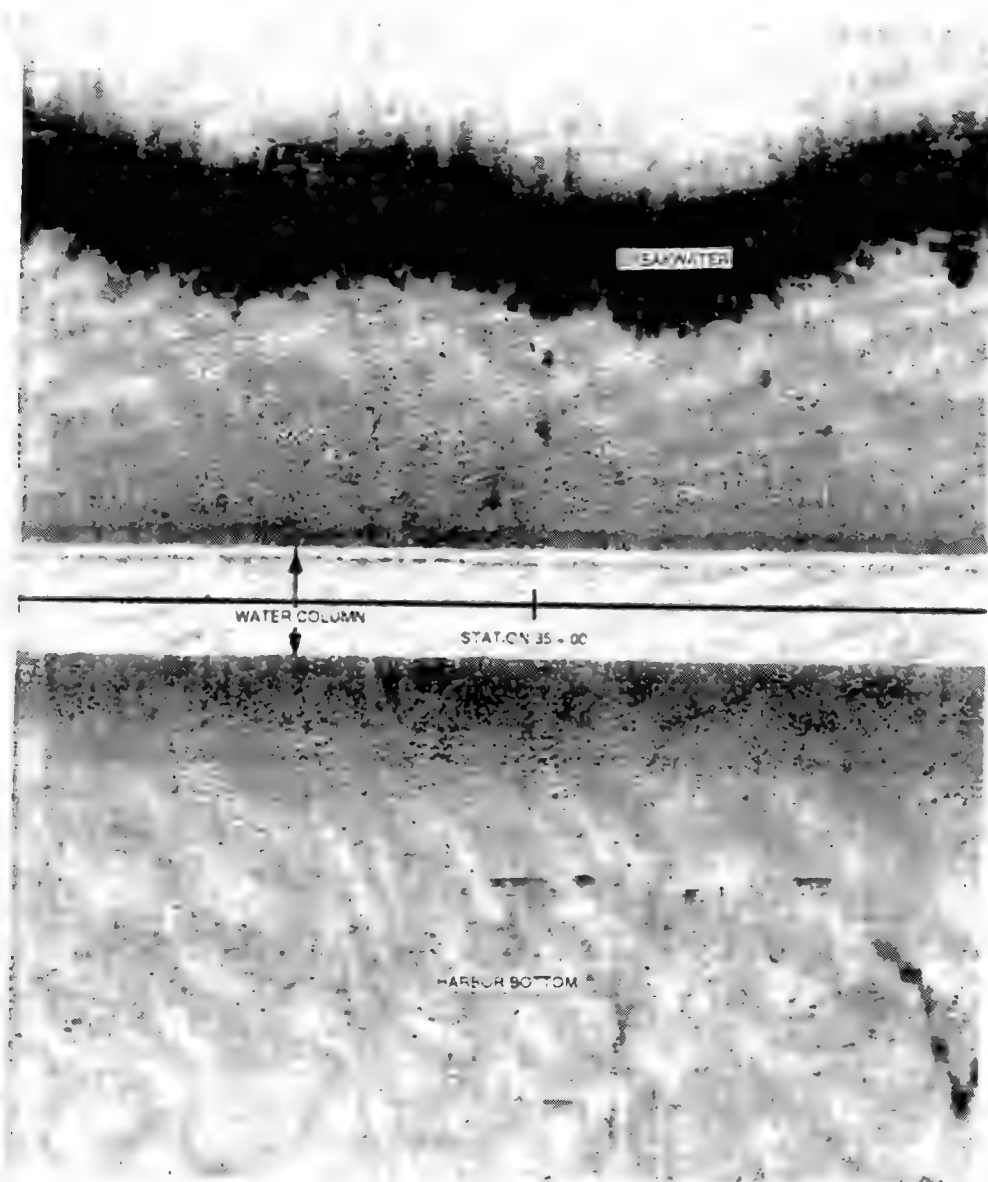
Figure 20. Typical portion of harborside area (SSS record, Station 30+00 to 40+00)

Water level measurements. Figure 27 is a plot of the hourly maximum and daily mean lake levels for the storm events described in Chapter 3. The long-term average lake level for Lake Michigan is +1.5 ft LWD. The U.S. Army Engineer District, Detroit, conducted a long-term study of Lake Michigan water levels, which was used to establish the water level versus return interval relationship for the Calumet Harbor water level gage.

Figure 28 is a plot of the mean depth above the gage at Site 1 and at the Calumet lake level gage during the February 8-9, 1987, storm event. The storm setup reached a maximum of about 0.8 m above the pre-storm level and persisted for about 24 hr at both sites. Note: Elevations at the two gages were
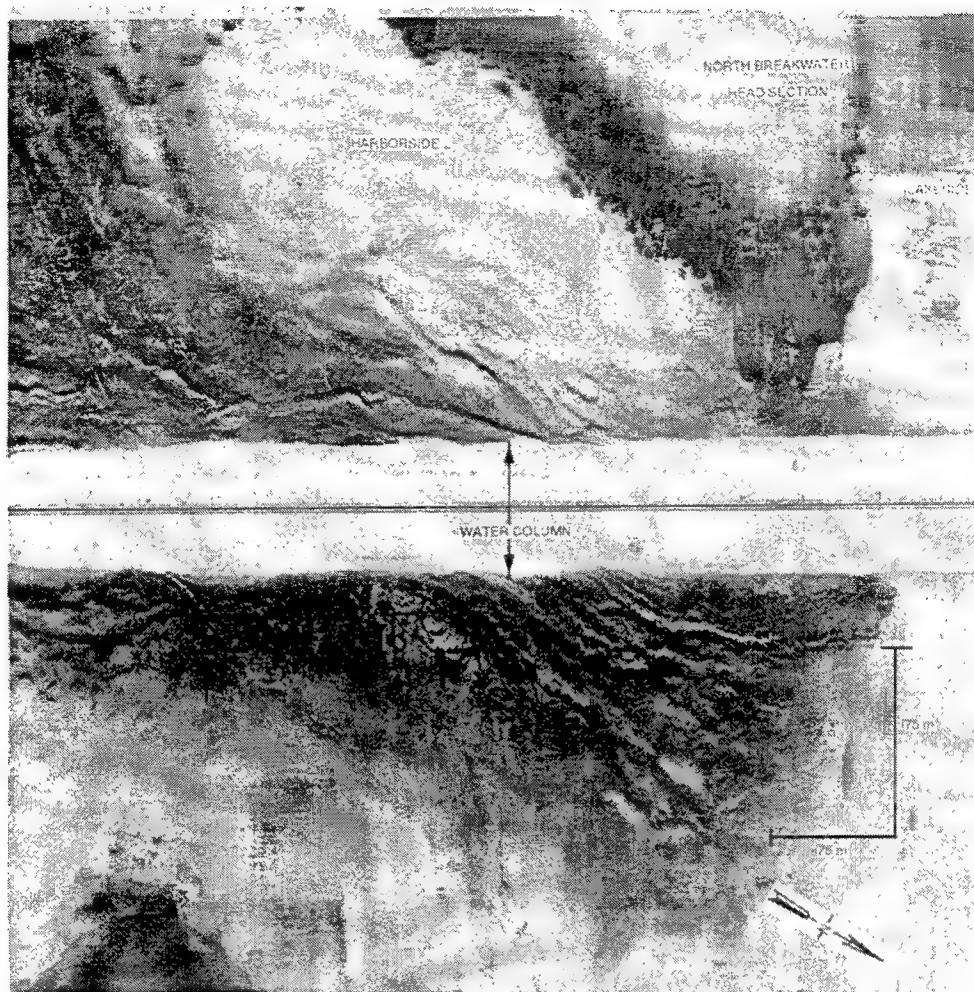
Figure 21. SSS record of scour features at harbor entrance

approximately correlated (visually) for Figure 28. A more rigorous statistical correlation would be required to accurately reference the depth data at Site 1 to LWD, as described earlier.

Data Analysis

Structure analysis

The structure analysis consists of three sections which are described in greater detail below: (a) structural condition analysis, (b) structural stability analysis, and (c) volumetric analysis.

1. **Structural condition analysis.** While a sudden increase/decrease in breakwater variability alone may not warrant attention, an increase in variance coupled with a significant change in cross-sectional area may indicate that an undesired process (damage) or desired effect (repair) is under way. In
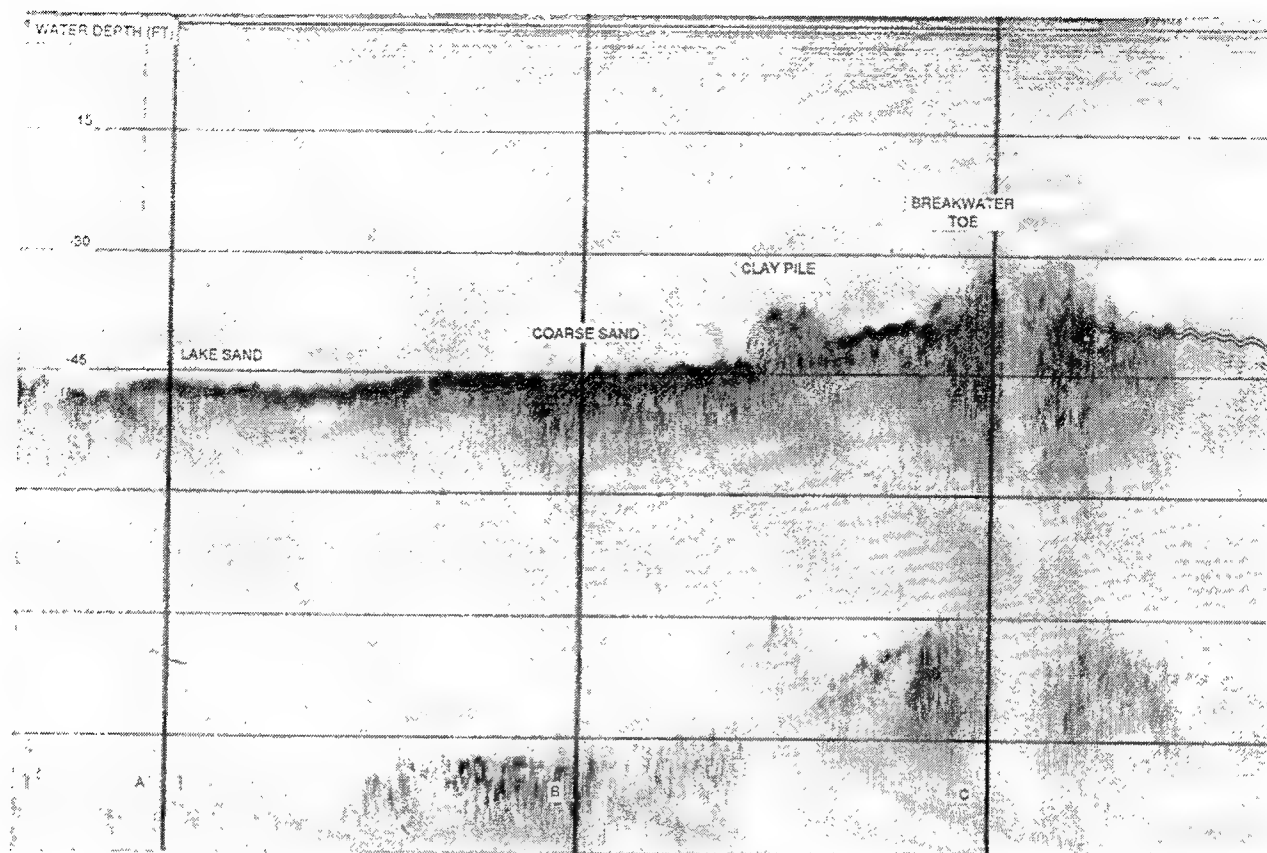
Figure 22. Subbottom sonar sonogram, line 25L

comparing variances (s^2) and mean cross-sectional areas (x) for all eight breakwater segments, the following four "trends" were identified:

Trend 1. Small variance change with a significant decrease in cross-sectional area is speculated to result from en masse breakwater settlement.

Trend 2. Large variance change with a significant decrease in cross-sectional area is assumed to result from numerous slope instabilities induced either by localized settlement or wave/ice damage.

Trend 3. Small variance change with a significant increase in cross-sectional area is speculated to result from successful widespread and uniform breakwater repair.

Trend 4. Large variance change with a significant increase in cross-sectional area is assumed to result from localized and nonuniform breakwater repair.
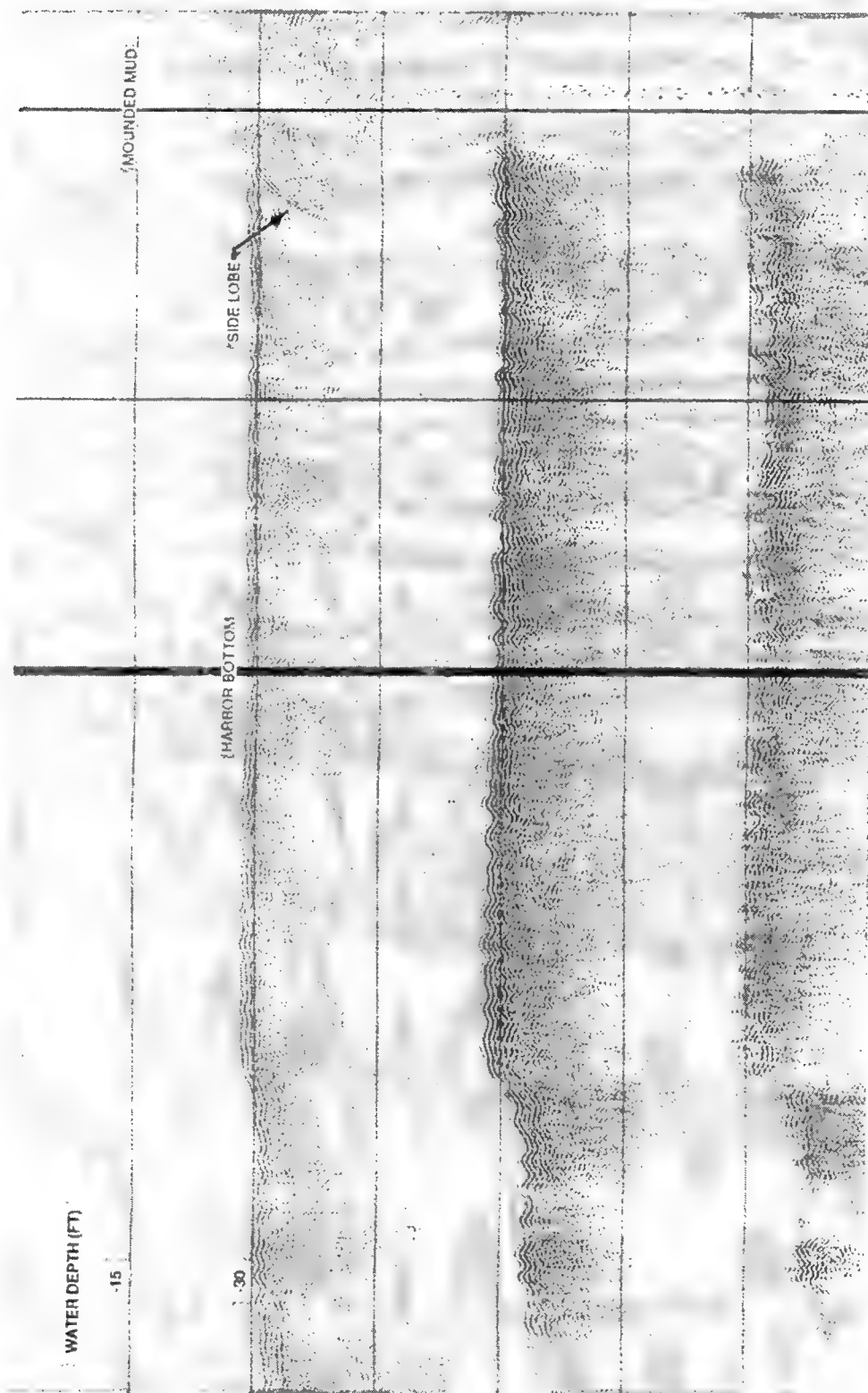
Figure 23. Subbottom sonar sonogram, line 35H
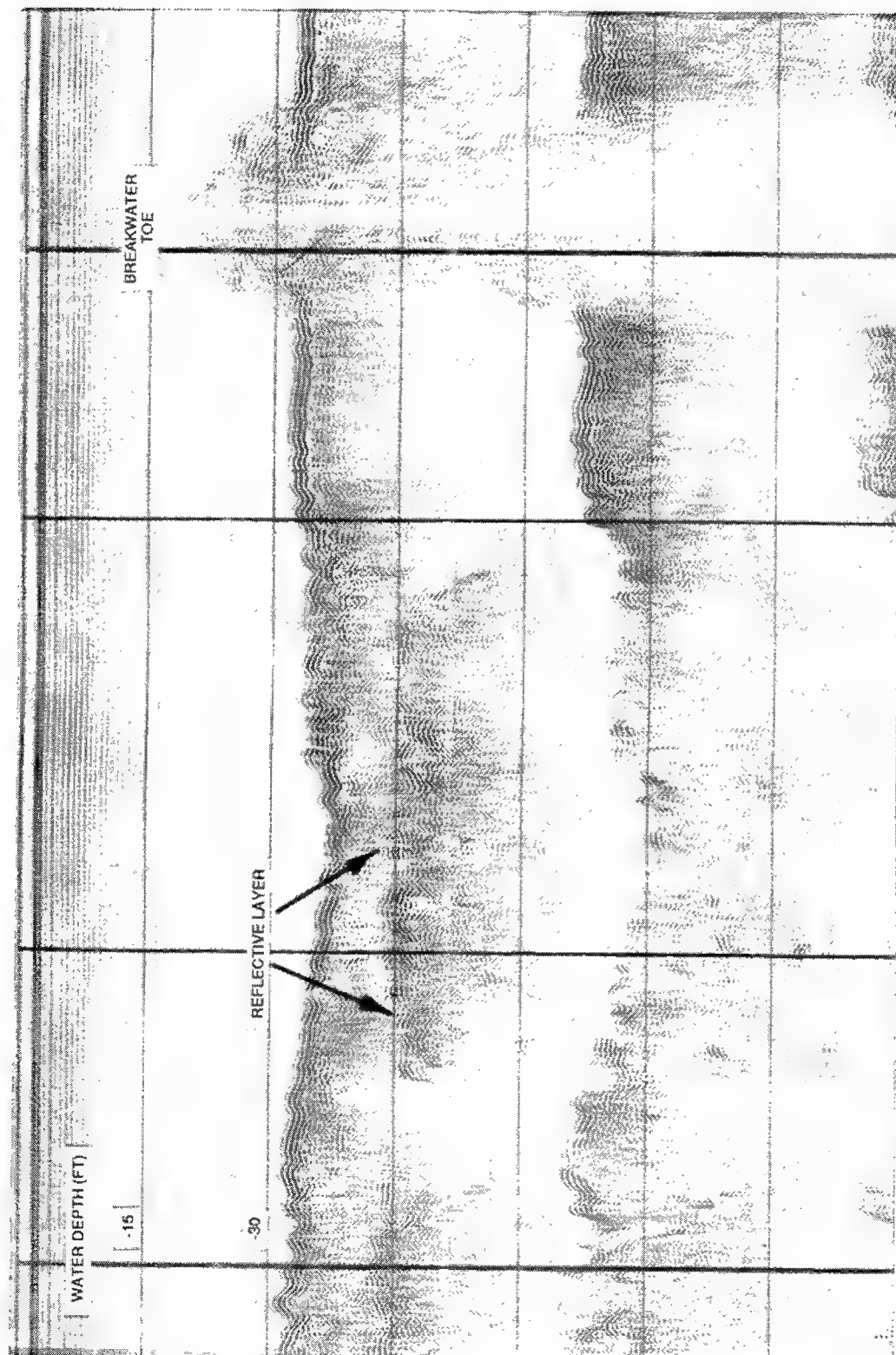
Figure 24. Subbottom sonar sonogram, line 6H
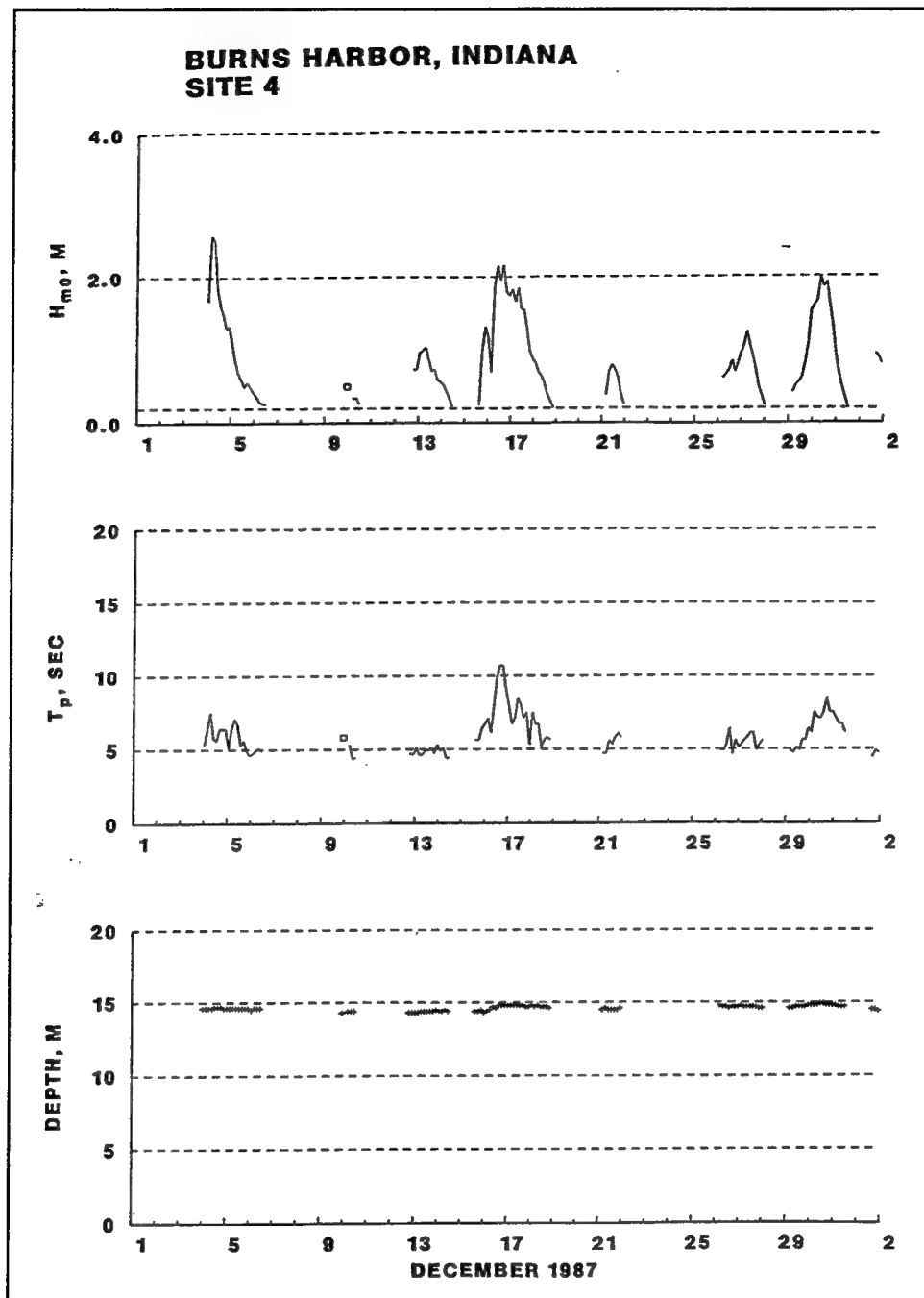
Figure 25. Sample wave data summary for Site 4, December 1987
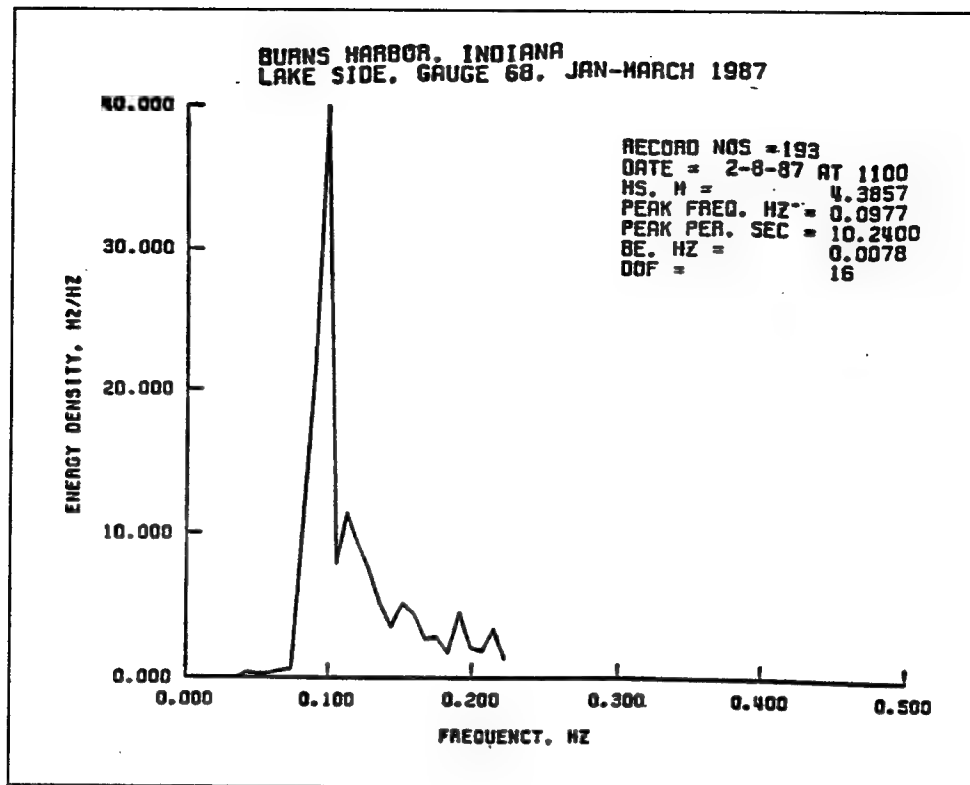
Figure 26. Typical sea surface energy spectrum for Site 1, 8 February 1987
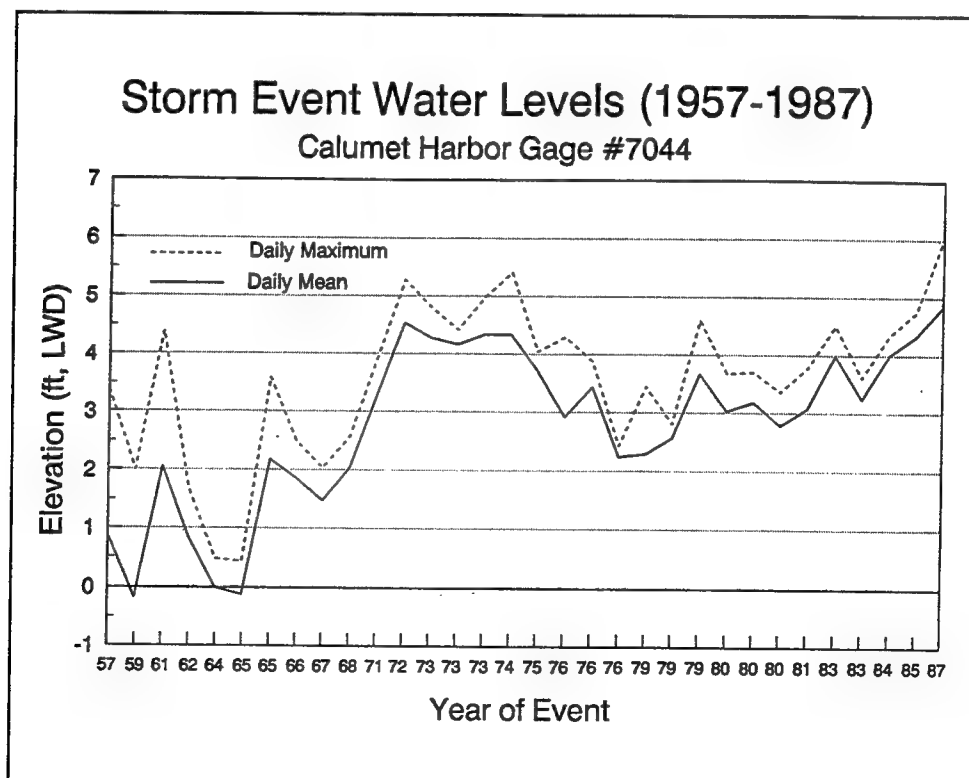
Figure 27. Maximum and daily mean lake levels for 32 hindcast storms
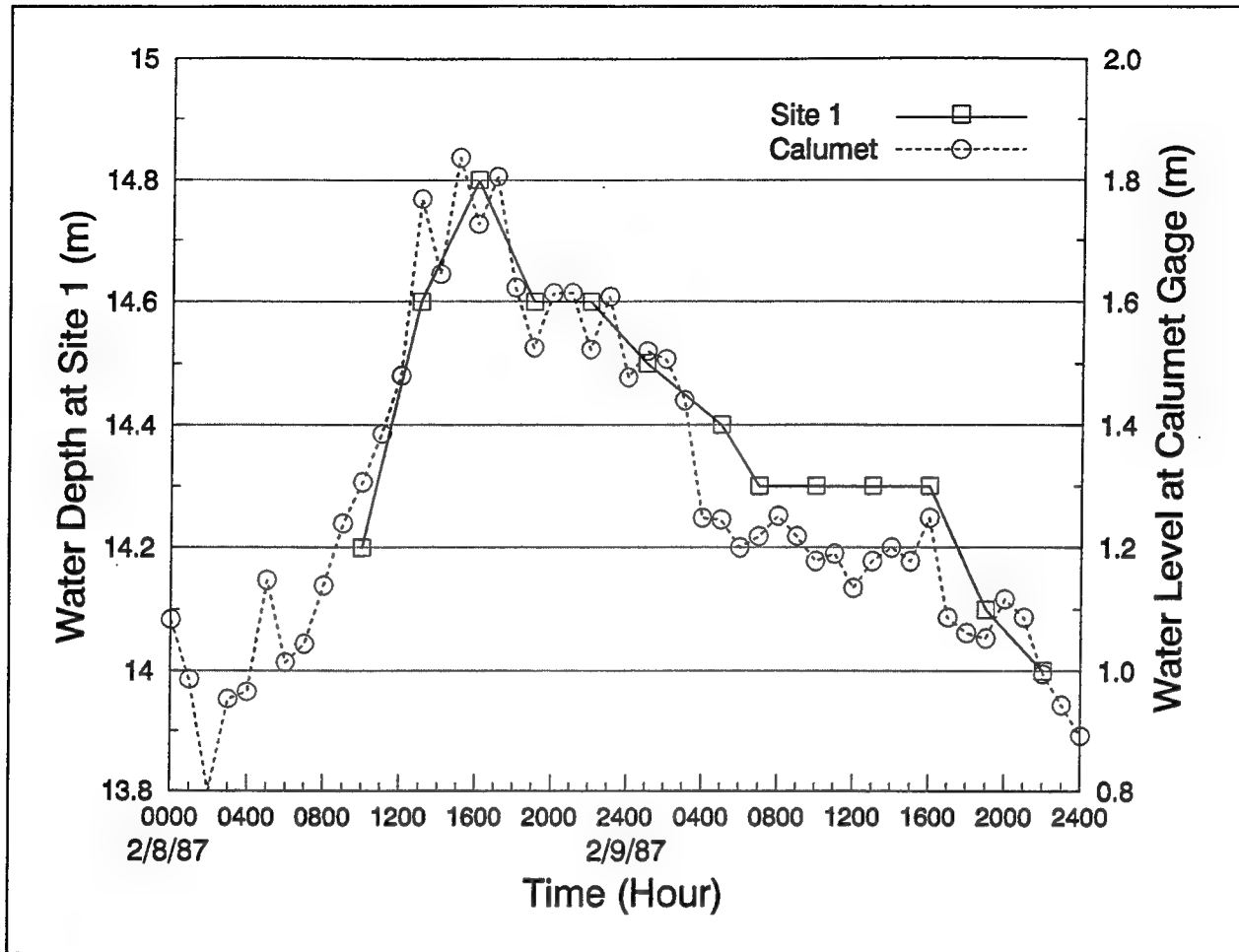
Figure 28. Wind-induced setup from February 1987 storm event, relative to lakebed at Site 1, Burns Harbor and to LWD at Calumet Harbor

Of the four possible breakwater responses, trend 2 is assumed to represent the worst-case scenario for breakwater damage. Conversely, the most desirable breakwater response would be trend 3. According to differences between the 1967, 1975, and 1989(92) surveys, breakwater segments 5 and 8 have experienced a trend 2 response. Segments 1, 3, and 4 show trend 3 response. Only segment 7 shows a trend 1 response, with segment 2 showing a trend 4 response. Segment 6 shows no significant change with respect to mean cross-sectional area or variance between 1975 and 1989, despite 17,600 tons of stone being placed on that portion of the breakwater for the same time period.

Actual changes in breakwater cross section. Actual changes in the breakwater cross-sectional area were determined for each breakwater segment by calculating the average change in area between survey-generated cross sections over a given time interval.

Positive changes in cross-sectional area due to maintenance were compared to area loss. By comparing three survey years (1967, 1975, 1989), temporal

aspects of breakwater cross-sectional change were used to identify specific problem areas. Details for the actual cross-sectional area change based upon breakwater surveys are described in Volume II, Chapter 5.

Location and magnitude of actual cross-sectional area change were calculated from the difference in area between time of construction (1967) and 1989(92). These temporal differences illustrate that while 5 of the 8 breakwater segments increased in size due to maintenance activity, segments 5, 7, and 8 decreased in size both in the upper and lower regions despite maintenance efforts.

Actual versus estimated cross-section change. 1967-75. Actual and estimated breakwater changes for 1967 to 1975 were minor. Due to the mild storm climate for this time period, wave damages were expected to be only 10 percent of the total experienced over the structure life. Settlement damages were expected to be 40 percent of the total. Actual and expected changes, in general, were shown to be within the detection threshold (random error of the survey).

1975-89. For 1975 to 1989, more significant changes were exhibited both in terms of damage as well as maintenance activity. Expected damages for this time period were 90 percent of the total wave damages and 60 percent of the total settlement damages.

1967-89. The final time period examined, 1967 to 1989, is the most comprehensive since it incorporates all of the information available. In terms of actual changes, five of the eight segments show a net positive area change ranging from +3 to +12 percent due to maintenance over the life of the structure. Mean expected changes followed the same trend as the actual changes for all segments. In magnitude, no segment had significantly less than expected damages, while 4 segments (1, 5, 7, and 8) showed greater than expected damages.

Assessment of cross-sectional area change. All segments/regions of the breakwater were constructed either at or above required specifications, according to the within- and across-segment comparisons for the 1967 survey. By 1975, however, the breakwater had experienced notable damage (diminished cross-sectional area) in some locations.

Upper breakwater regions within segments 2, 3, 4, 7, and 8 were considered deficient in cross-sectional area according to the across-segment comparison of the 1975 survey. In 1989(92), breakwater segments 1 through 6 were at or above the minimum cross-sectional area requirements. This is due to placement of 145,000 tons of maintenance stone during 1975-1989. However, segments 7 and 8 were significantly deficient in cross-sectional area as compared to the design template. The lower region of segment 5 was deficient in cross-sectional area.

Burns Harbor breakwater exhibited a highly irregular configuration for segments 2, 6, and 8 according to the 1989(92) survey. This is indicative of cumulative effects of random breakwater damage and incremental repair activities from 1975 to 1989.

2. Structural stability analysis

Actual damage. Table 4 lists the damage magnitude and residual for each breakwater segment. The residual is defined as the difference between actual and expected area changes. The expected area change is the sum of the (positive) area change due to maintenance stone addition and the (negative) area stone predicted to occur from both wave and settlement damages. The range in damage magnitude for each segment along the breakwater was from 10 to 21 percent. Taking the weighted average (according to segment length) of the damage magnitude, it was found that over the life of the breakwater there was a 14-percent damage magnitude. This means that 14 percent of the overall breakwater template required replacement due to damages.

Table 4
Breakwater Performance Parameters

Breakwater Segment	Area Change percent	Maintenance percent	Damage Magnitude percent	Residual percent
1	+12[1]	+31	19	-6
2	+5	+17	12	+2
3	+10	+20	10	+3
4	+11	+21	10	+1
5	-5	+9	14	-4
6	+3	+14	11	0
7	-12	+8	21	-9
8	-18	0	18	-7

[1] Values given are in percent of template.

Since the active armor unit zone is the part of the cross section which armors and protects the rest of the cross section, the percent damage to that area is important. A 14-percent damage magnitude can also be expressed as 111 percent of the active armor unit zone (using the lakeside armor zone only) or 66 percent of the active armor unit zone (using the lakeside, crest, and harborside armor unit zone).

Over a 21-year time period (1968 to 1989) these values (from Table 4) convert to 5.3 percent annual damage to the active armor unit zone (lakeside only) and 3.1 percent annual damage (lakeside, crest, and harborside), respectively. For the lakeside active armor zone only, the range in annual percent damage

would be from 3.8 to 7.9 percent damage per year. When the harborside and crest armor zones are included, the range in annual percent damage to the active armor unit zone is 2.2 to 4.7 percent.

Actual versus estimated wave damage. Calculations (see Volume II, Chapter 5) indicate that for the breakwater subjected to the WIS-simulated wave climate, a potential mean damage estimate of 60,450 tons of stone would be estimated over the 21-year life of the structure. This estimate represents 1.6 percent of the active armor unit zone or 2,880 tons average annual maintenance. Estimated average annual damages are less than half of the damages experienced.

Any attempt to quantify the damage for a given wave height above the design wave height produces a large variation in results, indicating that the wave damage process is very random. Losada (1991) found that scattering in wave damage results is greater for breakwaters constructed with regular-shaped armor units, such as parallelepiped blocks, than for structures constructed with irregular-shaped armor units, such as tetrapods or quarry stones. The range in potential damages represented by the variability of the stability coefficient is 0.15 to 8.5 percent of the active armor unit zone or 294 to 12,400 tons average annual maintenance.

Maintenance effectiveness and stone "loss." Cross-sectional change of the breakwater (presented in terms of stone quantities) was compared with the actual amount (tons) of maintenance stone placed on the breakwater, with the results shown in Table 5. An "N/A" entry is made for breakwater segments which did not experience statistically significant cross-sectional change between 1975 and 1989(92).

The breakwater's poor maintenance efficiency may be due to damage caused by waves/ice, foundation settlement, or both. In either case, maintenance/repair activities have not adequately addressed the breakwater's continual damage trend. Most of the damage sustained by the rubble-mound breakwater at Burns Harbor occurred from 1975 to 1989(92), which coincides with the fact that all maintenance for the breakwater occurred between 1975 and 1989.

3. **Volumetric Analysis.** Details of the volumetric analysis are found in Appendix C. Table 6 provides the combined change in tonnage (calculated from area change), by segment from the crest to the lakebed, between 1967 and the latest surveys. This change should be equivalent to the total amount of maintenance stone placed on the structure. A positive difference between the measured change and the maintenance stone, as in the case for area changes, is not physically possible. A small negative difference could also be the result of errors in the technique. A significant negative difference, however, indicates "lost" stone, which could only result (ignoring the possibility of theft) from settlement of the structure below the lakebed.

Table 5
Change of Breakwater Cross Section for 1975-1989(92)[1]

Segment No.	Length Lin. ft[2]	Estimated Amount of Stone Gained/Lost[3]			Actual Amount Placed Net Total tons[5]	Difference/ft est. - act. tons/ft	Rank
		Upper Region tons	Lower Region tons	Net Total tons			
1	600	+10,526	+4,180	+14,737	34,524	-33.0	1
2	900	+9,124	N/A (0)[4]	+9,932	24,556	-16.2	7
3	500	+7,528	N/A (0)	+8,042	15,943	-15.8	8
4	800	+9,616	+3,148	+12,764	26,301	-16.9	6
5	500	N/A (0)	-4,727	-5,497	7,313	-25.6	2
6	800	+4,072	-1,882	+2,156	17,585	-19.3	5
7	300	N/A (0)	-3,067	-3,952	3,510	-24.9	3
8	600	-3,388	-9,393	-13,038	0	-21.7	4

[1] Differences are in terms of stone lost or gained per breakwater segment and are given in lineal foot of breakwater; lower numbered rank designates a more severe stone loss/attrition rate.

[2] 100-ft "buffer" zones separating breakwater segments are not included.

[3] Determined from change in average area of cross-sectional region within the segment of interest based on: average area difference (1989(92) - 1975) × segment length × 0.59 × 145 pcf/2,000 pounds per ton. For segment 1, 0.65 was used for percent solids and 160 pcf was used for specific weight.

[4] N/A = Statistically no change in segment cross-sectional area for a particular region.

[5] Maintenance stone placed in 100-ft "buffer" zones not included.

Table 6
Tonnage Change (1967 - 1989(92))

Segment Number	Measured Tons	Maintenance Tons	Difference Tons	Equivalent Settlement, ft
1	13,333	18,045	- 4,712	-0.7
2	6,206	27,294	-21,088	-2.3
3	9,026	19,246	-10,220	-1.9
4	16,306	27,471	-11,165	-1.4
5	-4,413	8,483	-12,896	-2.4
6	4,568	19,287	-14,719	-1.8
7	-9,069	3,627	-12,696	-3.5
8	-20,282	234	-20,516	-3.2
Totals	15,675	123,687	-108,011	

When maintenance stone is considered in the mass balance, every segment is significantly deficient. The total amount of deficient stone, about 108 kt, is about the same order as the 144 kt of maintenance stone placed. For comparison, the total amount of armor stone called for in the General Design Memorandum for the entire structure is 225 kt.

For a first order estimate of the amount of settlement this represents, all of the "missing" stone was assumed to have been uniformly placed along the breakwater section, both lake and harborside, and an equivalent settlement calculated assuming the structure moved downward monolithically. These values (Table 6) range from 0.5 to 3.5 ft, with a weighted average (by segment length) of 2.1 ft for the entire structure.

Geotechnical analysis

Sediment samples. No lakebed sediment like Sample 3, the coarse sand and gravel, was identified in the pre-construction sampling. There is no known natural process that would distribute discrete patches of coarse sand over finer sand in this environment, and their coincidental proximity to the breakwater is not likely; therefore, the coarse sand is assumed the result of breakwater construction activities.

Coarse sand was used in backfilling the excavated foundation and for the core of the structure. It is possible the sand was removed from the core after construction and deposited in the piles by some unspecified interaction between incident and reflected wave forces. This is not considered likely, because the sand is distributed longitudinally in discrete piles. The transverse

extent of the piles is over 300 ft (Figure 18) and covers both nodal and antinodal positions for the longer waves.

A clue to the origin of these sand piles is provided by the position of the adjacent clay piles. Examination of Figure 8, the foundation excavation history, reveals that in both 1966 and 1967, significant portions of the trenches were excavated in the fall and not backfilled with sand until the next spring. Over 2,000 linear feet of trench remained open through the winter. The remnant clay piles show the excavated material was dumped lakeward of the trenches. It seems reasonable to assume that some portion of this excavated material was re-deposited in the open trenches by wave action over the winter. If the significant volume of this re-deposited clay was in the trench bottom when backfill operations commenced, the design level of the backfill would be reached before all the sand contracted for was used. Disposal of the "extra" sand lakeward of the structure may have been the most economic alternative.

The aerial extent of the piles is about 90,000 yd^2. Reliable estimates of the volume of sand in the piles would require extensive borings to determine their thickness. The SBS survey did not reveal a discernible interface below the sand piles, but the difference in acoustic impedance between the coarse sand and the finer lakebed sand is likely too small to provide a strong horizon on the sonogram. While on an inspection dive, the author probed several piles by hand and found thicknesses greater than 3 ft, but this is not considered conclusive. It seems reasonable that the thickness is on the order of a foot or more, since a thin veneer of coarse sand (on the order of the ripple heights) would not be durable enough to remain distinct over the ensuing years. Therefore, an estimate of the volume present in the piles is 30,000 to 90,000 cu yd. This is in the same range of estimates as the volume of excavated trench exposed over the winter. The total volume of sand backfill placed in the trenches, according to construction progress reports, was on the order of 400,000 cu yd.

Settlement recalculation. Reanalysis by the GL is reported in Volume II, Chapter 4. This reanalysis used the same input soil conditions from the pre-construction test borings. Results were similar to the original predictions of settlement for the unprepared foundation. Estimates of settlement for the foundation as designed range from 1 to 2 ft, with an average of 1.5 ft. Using the inverse of the equivalent settlement definition used in the "Structural Stability Analysis" section, this represents a loss of about 73 kt of armor stone.

A comparison of changes in the crest elevation over time showed minimum deflection coinciding with the locations of maximum clay removal, but because of the scatter in the data and the problems associated with determining settlement from the survey data, this result is not considered conclusive.

In addition, settlement was predicted for two hypothetical cases to evaluate the impacts on settlement if the excavated clay was redeposited in the open trenches prior to backfilling with sand, as postulated above. The two cases assumed uniform distribution of 5 and 10 ft of "remolded" clay in the trench under the sand backfill. Various physical properties of this remolded clay

were also assumed. The resulting settlement was predicted to range between 1.5 and 2.5 ft. Most of this settlement would have occurred rapidly during the construction phase (viscoelastic settlement), with a much smaller rate of consolidation settlement occurring over the life of the structure. Since the initial settlement would have resulted in additional material being added during construction, it is not likely these two modes of foundation settlement would have been a factor in any longer-term crest loss. The conclusion of Volume II, Chapter 4 states:

> Original settlement calculations of the...breakwater structure were confirmed...Settlement during and shortly after construction probably occurred essentially as expected, and is therefore not apparent in post-construction surveys...Major settlements of the breakwater crest... are less than 5 ft...No effort was made to calculate observed settlement after 1975, when significant amounts of repair stone were added to the structure. It is unknown that settlement has played any significant role in the unsatisfactory performance of the breakwater.

A second reanalysis of the as-designed foundation was performed by the NCC Geotechnical Coastal Branch that included the effects of the maintenance stone. Using the estimated settlement areas and their applicable lengths along the breakwater, the potential change in volume and tonnage of stone were calculated. Table 7 lists the estimated loss of cross-sectional area due to settlement, by segment. The result is a somewhat lower total settlement tonnage loss than the 73 kt calculated from the GL settlement figures, corresponding to 51 kt of armor stone.[1] However, this is the ultimate predicted settlement, and estimates of the rate of settlement were also made. The portion of ultimate settlement at the time of the 1987 (89) surveys ranges from 35 to 75 percent, depending on whether the escape of the water in the soils is modeled as one- or two-dimensional drainage. Since experience suggests that the consolidation theory used in this analysis represents an upper limit, it was assumed that essentially 100 percent of consolidation had occurred by 1993. The settlement distribution across the section in the NCC study is modeled (possibly more realistically) as trapezoidal, with minimum settlement at the toe and maximum settlement at the crest. For purposes of comparison with the GL prediction and the observations, 51 kt corresponds to an equivalent vertical (monolithic) settlement of about 1 ft. The NCC study concludes that "... the amount of missing stone is related to the expected settlement of the breakwater." Additional information on the NCC geotechnical settlement reinvestigation is provided in Volume II, Appendix 5C.

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[1] The NCC estimate of 60.8 kt is based on the average density of the structure (1.37 tons/yd^3 as opposed to average density of the armor layer only (1.15 tons/yd^3)).

Table 7
Estimated Cross-Sectional Area Loss Due to Settlement

Breakwater Segment	Segment Length	Potential Settlement		Area Loss, ft²		
		Center Line ft	Toe ft	Minimum	Mean	Maximum
1	650	1.08	0.32	87	173	260
2	1000	1.90	0.58	152	304	456
3	600	1.66	0.38	129	257	386
4	900	1.03	0.33	83	166	249
5	600	0.80	0.23	64	127	191
6	900	1.04	0.33	84	167	251
7	400	1.21	0.42	99	197	296
8	700	1.43	0.43	114	228	342

Wave and water level analysis

Wave reflection/transmission. Figure 29 shows the incident plus reflected energy-based significant wave height (Site 1) plotted as a function of the simultaneously measured incident wave height (Site 4) for incident waves over 0.5 m when hindcast winds were from the northern quadrant. Results of the analysis show that reflection for low waves, less than about 2 m, was essentially negligible, but was significant for waves above this level. The actual reflection is not, of course, a step function at 2 m, but this value serves as a useful and convenient threshold, since smaller waves are unlikely to have any impact on the structure.

The reflection coefficient K_R is defined for monochromatic waves as:

$$K_R = \frac{H_R}{H_I} \tag{1}$$

Since the measured values of significant wave height are based on an energy-derived parameter, an assumption is made that energy is conserved; i.e.,

$$(H_{I+R})^2 = H_I{}^2 + H_R{}^2 \tag{2}$$

leading to an equivalent expression for K_R

$$K_R = \left[\frac{(H_{I+R})^2 - H_I{}^2}{H_I{}^2} \right]^{\frac{1}{2}} \tag{3}$$

Figure 30 plots the square of wave heights at Sites 1 and 4 for incident waves over 2 m. A least squares fit to the data is also plotted and corresponds to

$$(H_{I+R})^2 = -1.08 + 1.39 H_I^2 \tag{4}$$

The 90-percent confidence bands for Equation 5 are also shown on Figure 30, but it is debatable how meaningful that statistic is for such a small sample.
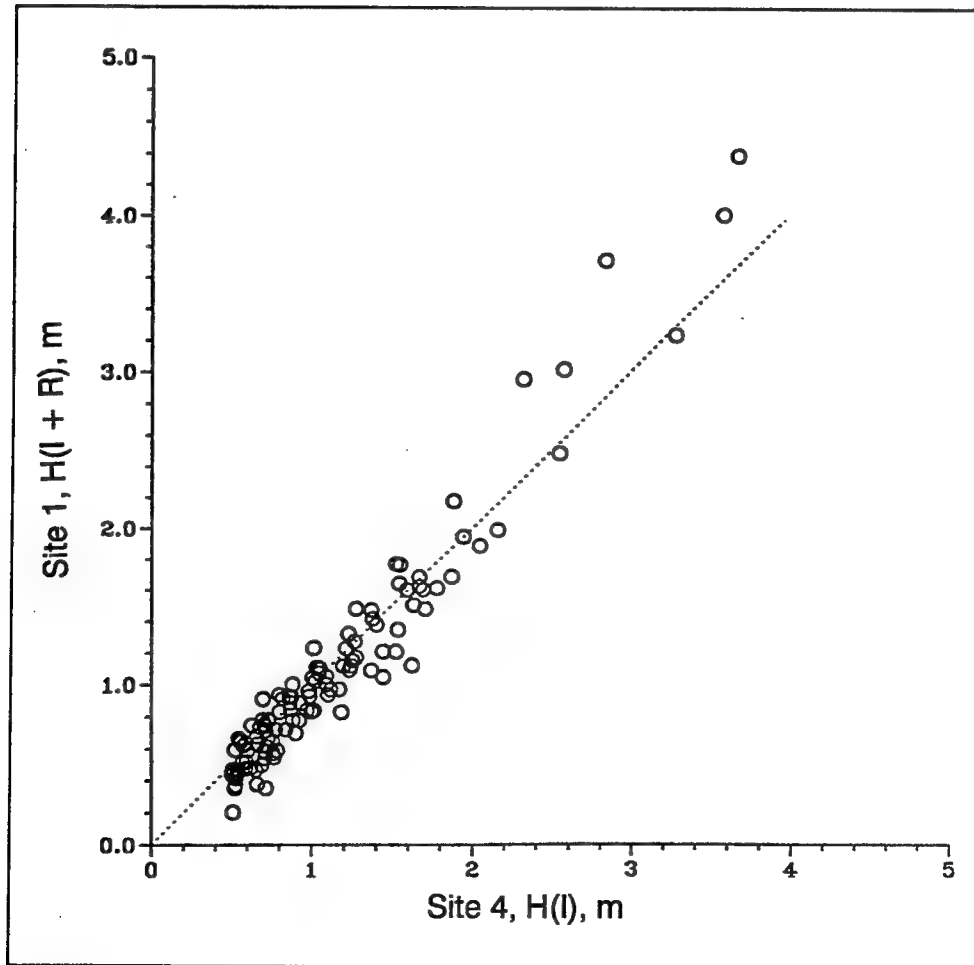
Figure 29. Significant wave heights (*Hs*) at Site 1 versus Site 4
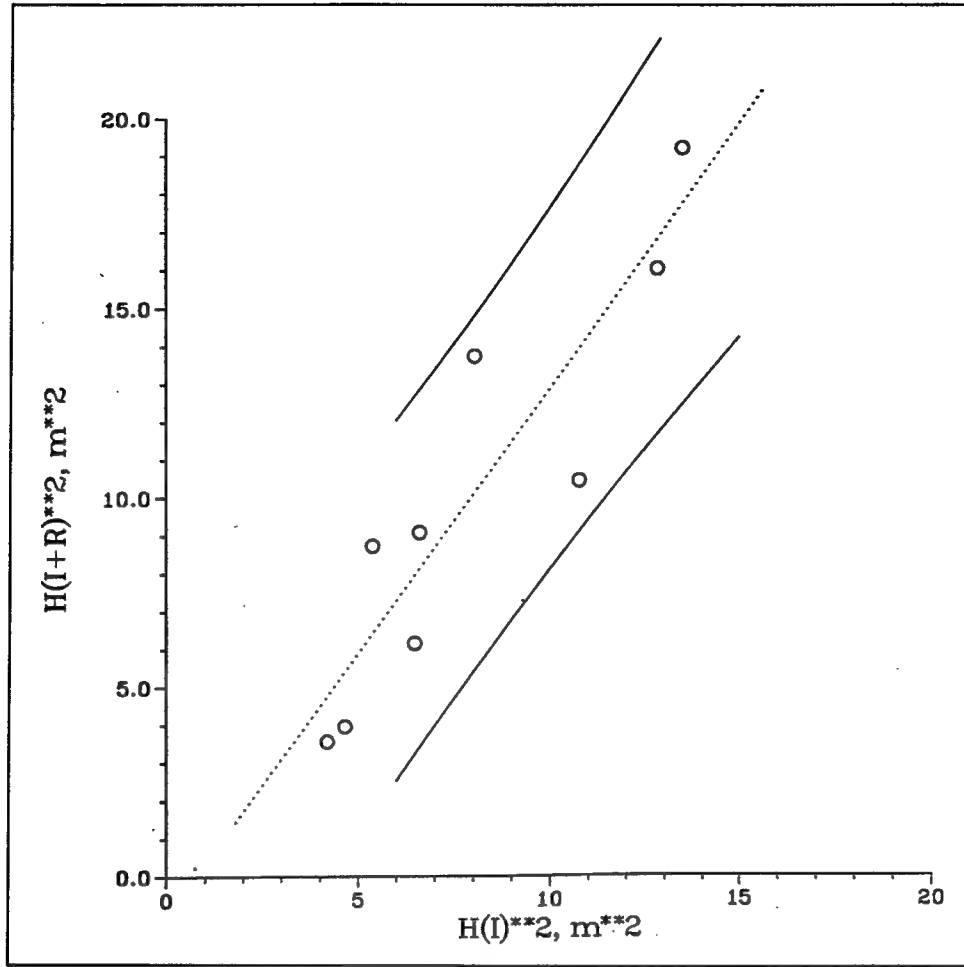
Figure 30. Least squares regression of significant wave height squared at Site 1 versus Site 4 for *Hs* > 2.0 m

Visual inspection confirms that measured values are within about a meter of the predictor equation. The resulting reflection coefficient K_R is plotted in Figure 31 as a function of incident significant wave height, and is approximated by the equation

$$K_R \cong \begin{cases} 0 , & H_I \leq 2.0 \ m \\ 0.62 \sqrt{1 - \dfrac{2.77}{H_I^2}} , & H_I > 2.0 \ m \end{cases}$$

(5)

For the wave transmission analysis, wave heights measured at Site 1 were adjusted by the calculated reflection relation given in the previous equation. These calculated incident wave heights (corresponding to hypothetical Site 1A) were used in conjunction with simultaneously measured transmitted wave heights (Site 2) to calculate transmission characteristics. The results are
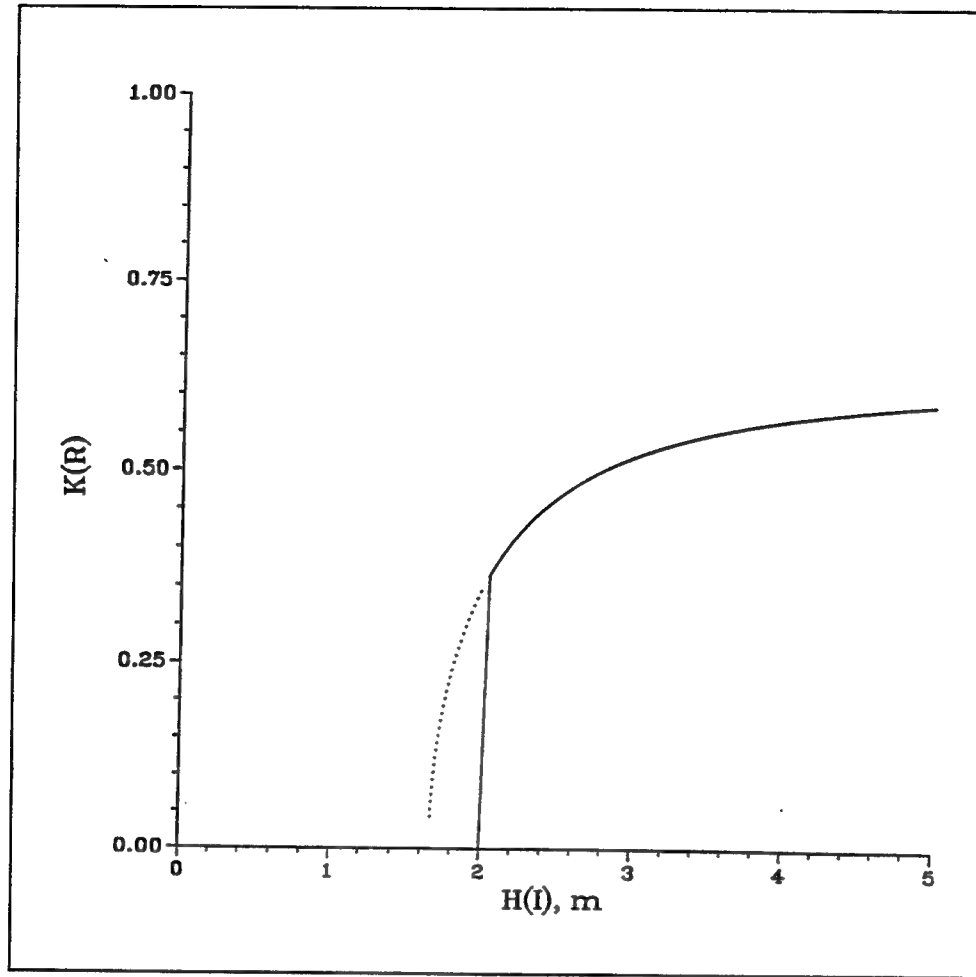
Figure 31. Reflection coefficient *K(R)* versus incident significant wave height

presented in Figure 32 where K_T is plotted as a function of incident wave height, and is represented by the equation

$$K_T = 0.192 - 0.052H_I + 0.018H_I^2 \tag{6}$$

Figure 33 is another plot of K_T, but plotted against wave power. The scatter is reduced compared to Figure 33, partly due to better representation of higher waves with short periods that have little impact on the structure. Because both theory and the measurements showed a dependence of K_T on wave period as well as height, and wave power incorporates both parameters, correlations based on wave power may be a more useful way of defining and calculating transmission for porous structures.

Figure 34 is a comparison of the prototype and the 2-D model transmission coefficients. The prototype data ranged from 4.1 to 11.6 sec, but only those prototype waves with periods greater than 10 sec, comparable to the 11-sec model waves, are plotted. The 2-D model measured wave transmission at two

62
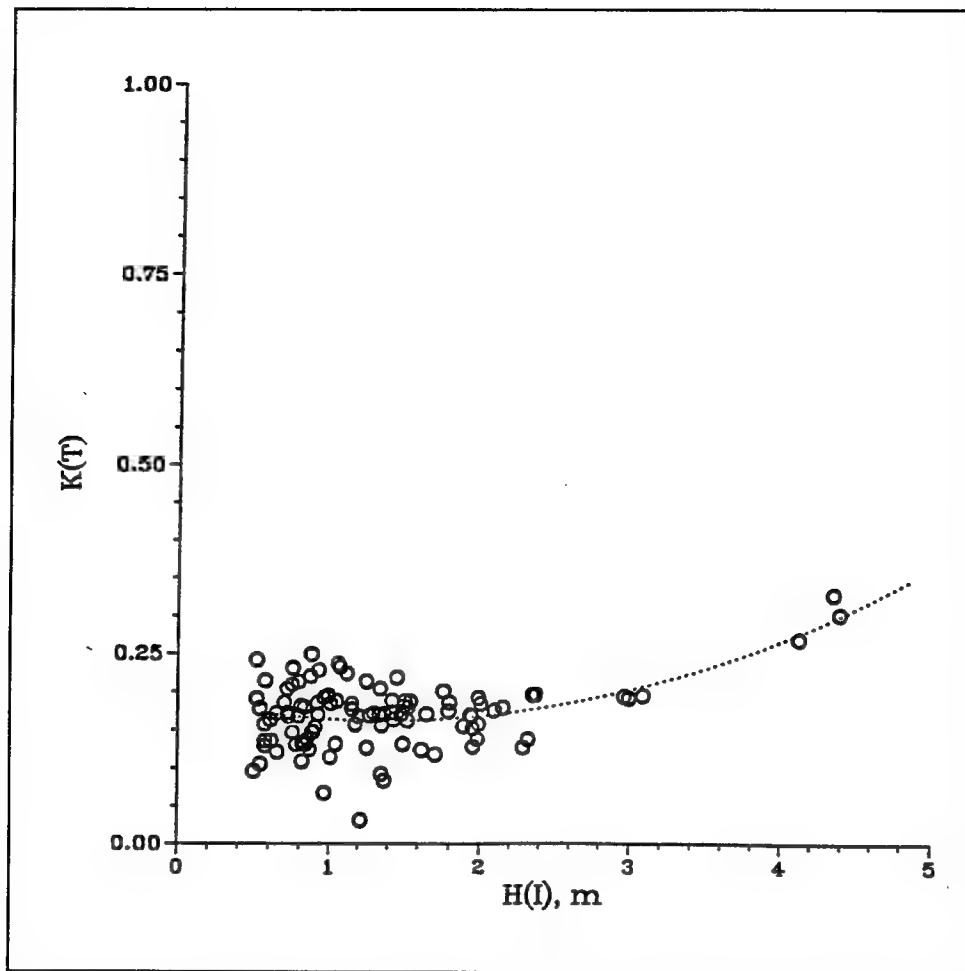
Figure 32. Transmission coefficient *K(T)* versus incident significant wave height

locations at distances $L/2$ and L behind the breakwater, where L was the shallow-water wavelength. Transmitted wave heights were generally higher at the L position than the $L/2$ position. The transmitted wave measurements in the prototype were made at a fixed distance of about 75 m from the center of the breakwater. This location falls halfway between $L/2$ and L for the longer waves of interest (L on the order of 100 m). It is not certain that the increasing trend with distance behind the structure exists in the prototype, so both the L and $L/2$ model data sets are plotted.

Only eight prototype incident wave data points meet the criteria of having periods greater than 10 sec, and these are compared to 10 modeled incident wave heights. To clarify the effect of applying the reflection correction to the measured data at Site 1 (and because there is no documentation on the treatment of reflected waves in the model), the uncorrected transmission coefficient $K_T' = H\ (Site\ 2)/H\ (Site\ 1)$, is also plotted. The general trend of all four data sets is increasing transmission with increasing wave height, though the prototype data from Site 1A reveal highest transmission, with a maximum of 0.33
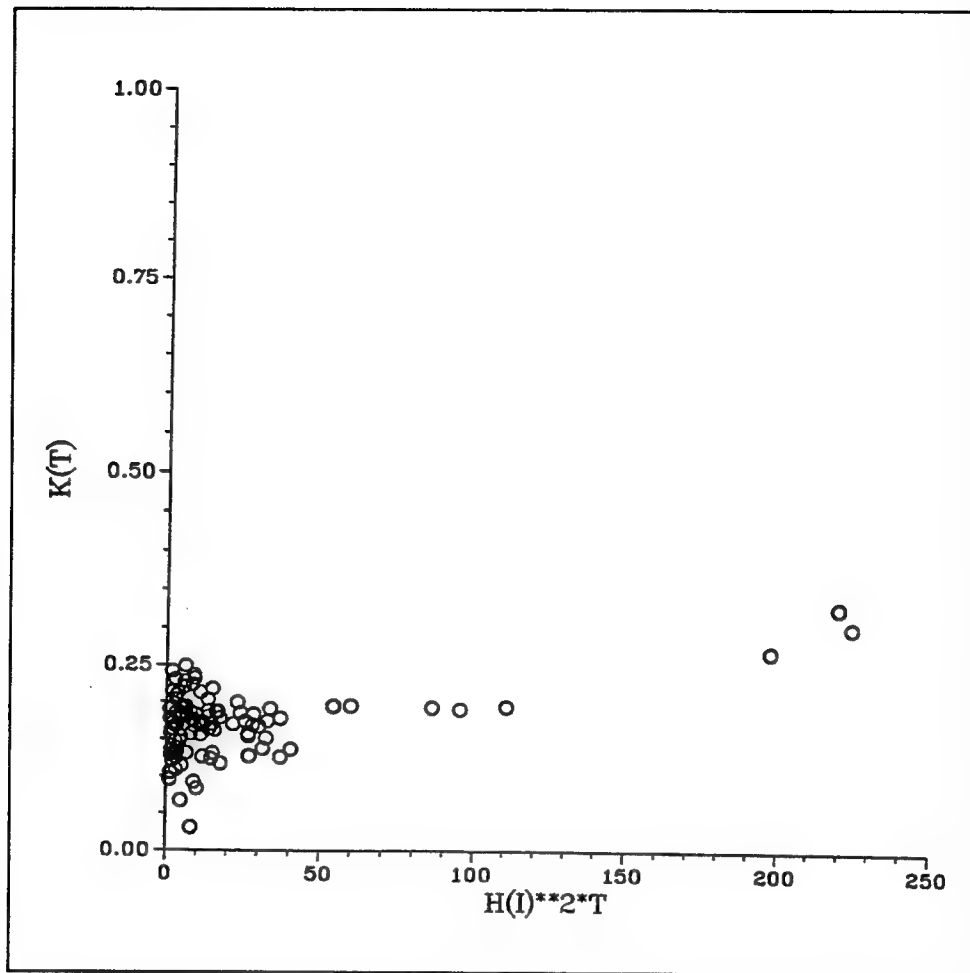
Figure 33. Transmission coefficient *K(T)* versus incident significant wave power

for the 4.35-m incident wave; the model reaches 0.32 for the 5.5-m wave. Data from Site 1 more closely follow the model results.

The model significantly underpredicted transmission for waves below 3 m, regardless of whether the reflected waves are included. Long-period waves below 3 m are transmitted through the structure without overtopping, and are strongly influenced by the structure's porosity. Results of later research showed that the core material in scaled physical models should be oversized relative to the linear scaling relationship to compensate for viscosity effects (Keulegan 1973). The core material in the 1966 study was sized linearly, like the cover layers, and underprediction may result from the effect of the increased viscous drag, relative to the prototype, at these scales.

Another factor that would tend to increase the measured energy at Site 2, and thus the prototype transmission coefficient, is the effect of energy coming through the entrance, in spite of the attempt to minimize this influence by its position. Finally, the lake level during the more extreme events exceeded the 4-ft LWD used in the model study. Measured data for waves over 3 m are
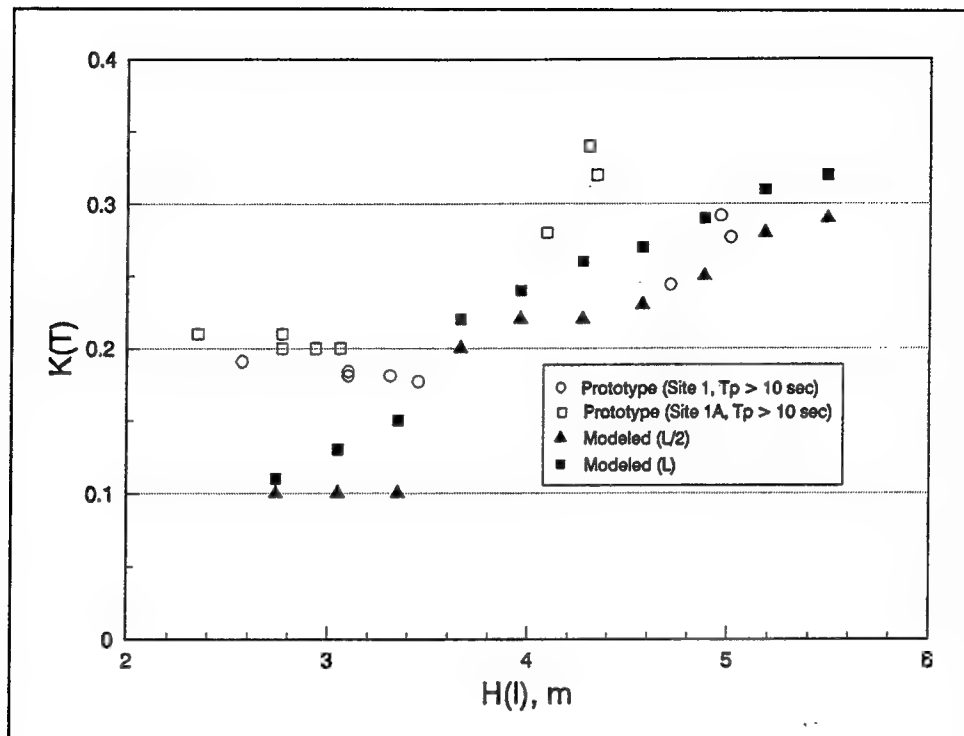
Figure 34. Prototype and modeled transmission coefficient $K(T)$ versus incident significant wave height when $T_P > 10$ sec

from storms that occurred on February 8-9, 1987, and March 9, 1987. Lake levels for the February storm, as measured at the Calumet Harbor gage, exceeded 1.8 m (6 ft LWD). This increased water level undoubtedly affected the transmission. The lake level at Burns Harbor during the March event, as measured at Site 1, was very near the 4.0-ft LWD used in the model study.

Evaluation of the model's performance presupposes that the model cross section duplicates the prototype; i.e., that the actual structure was constructed as designed. Stability analysis has shown that significant amounts of armor have been added to the structure without a concomitant increase in structure elevation or volume. Therefore, the existing structure must contain a higher percentage of armor, and the less porous layers must be correspondingly lower in the cross section than the design structure. Whether this increased porosity is sufficient to account for increased transmissivity cannot be determined with the existing data, but it is certainly a contributing factor.

The model data show an abrupt discontinuity around 3.5 m. It is near this point that the model study indicated overtopping occurred. It seems likely the additional energy coming over the model structure caused the increase in total transmittance. There is not as obvious a jump in the prototype data, though it could be argued that an increase in the rate of transmittance occurs between 3 and 4 m. It is likely that this corresponds to the onset of significant

overtopping in the prototype as well. The model predicts total transmittance better in the combined transmission/overtopping regime.

Direct comparison of the model and prototype is hampered by the following factors:

a. Model data.

    (1) Regular waves.

    (2) Uncertainty in incident wave height (uncertainty about correction for reflection).

    (3) Scale effects (core sizing).

b. Prototype data.

    (1) Irregular waves.

    (2) Uncertainty in incident wave height (on the order of 1 m).

    (3) Peak periods different from model wave periods.

    (4) Transmitted gage position different from model.

    (5) Uncertainty in actual cross-section composition.

    (6) Uncertainty in mean water levels during storms.

Wave conditions at the grain dock. Site 3 is in front of the sheet-pile grain dock, and the waves show the effect of the reflected energy. The wave height measured in front of the wall will depend upon the position of the gage and frequency of the incident energy. If the incident wave is approximated by an equivalent monochromatic wave, and if the wall is 100-percent reflective, a clapotis (standing wave system) with a double amplitude (height) ratio to the incident wave of

$$
\frac{H3}{H2} = 2\cos\left(\frac{2\pi x}{L}\right) \tag{7}
$$

will form.

Figure 35 is a plot of the ratio of measured wave heights at Site 3 to Site 2 against the peak period data at Site 2. Equation 7 is also plotted on Figure 35 for $x = 45$ ft, the distance of Site 3 from the front of the grain dock. L was estimated from the linear shallow-water approximation for a depth of 30 ft. For example, for 6.7 ft, the highest wave measured at Site 3, Equation 7 predicts an incident wave of 4.8 ft, which agrees well with the 4.7-ft wave height

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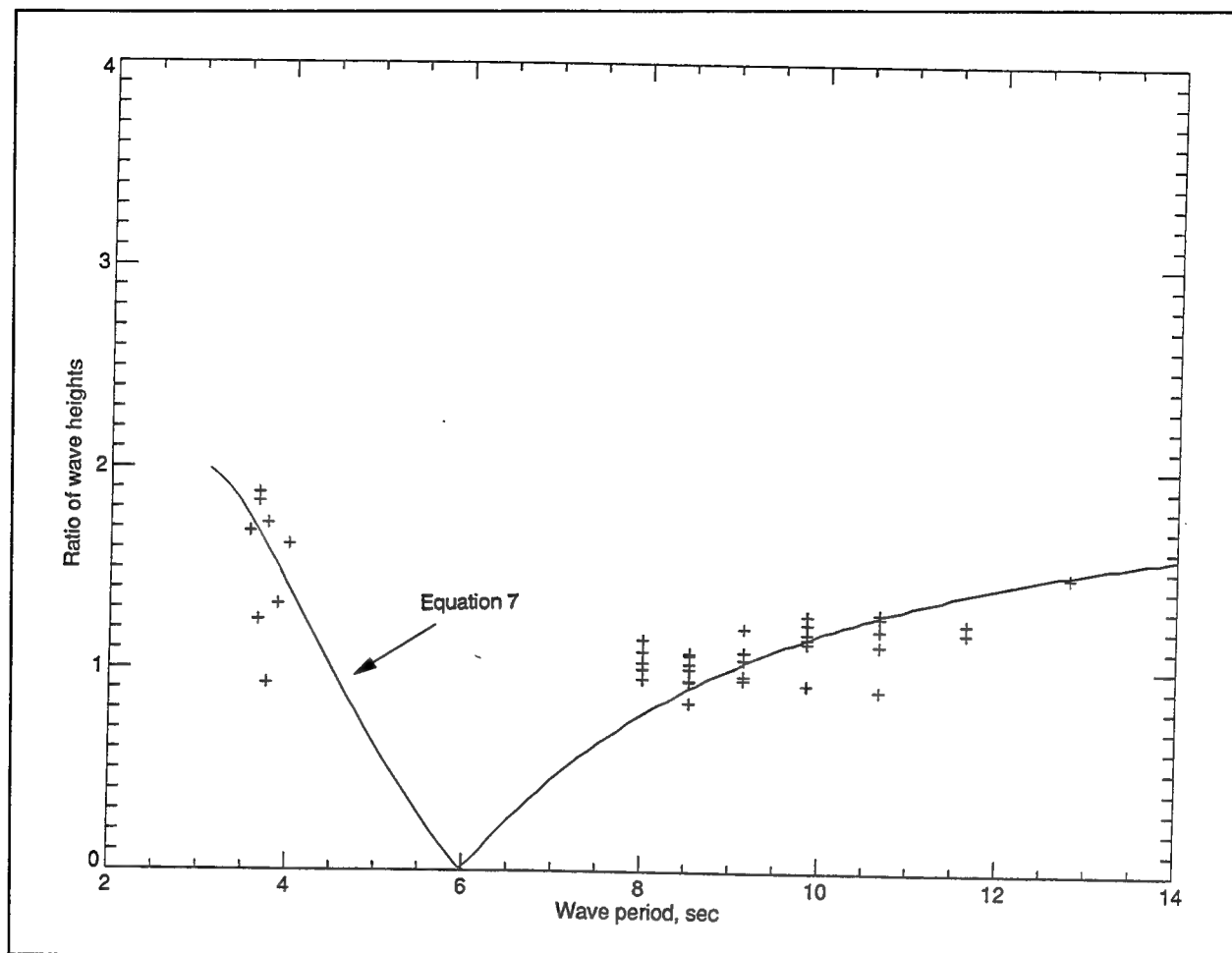
Figure 35. Ratio of significant wave height (Site 3 to Site 2) versus peak period (Site 2)

measured at Site 2 at the same time. This indicates wave conditions at the grain dock result from complete reflection of energy transmitted through the breakwater.

Harbor seiching. Visual examination of energy spectra plots did not reveal any significant energy at frequencies below 0.05 Hz, corresponding to waves longer than 20 sec, in either the incident or transmitted data.

Extremal analysis. Two sources were available for use in describing the incident wave climate: the measured data from NDBC Station 45007, and the hindcast data for WIS (Station 64, the nearest station to Station 45007, and Station 62, located 10 nm seaward of Burns Harbor in 22 m of water). The measured data are considered more accurate, but are of shorter duration (statistics are calculated from 1973 through 1989) and intermittent (the buoy is removed each winter to avoid ice damage). Direct comparison of their results is hampered by differences in their reporting formats and in their wave height and period discretization bins, and by the lack of directional information and winter measurements by the buoy. However, comparison of the frequency of

occurrence of storm wave heights from these two sources revealed some differences.

The occurrence of waves higher than 3.5 m for WIS Station 64, when summed for all directions, is 0.22 percent. The average percent occurrence of waves over 3.5 m for the NDBC buoy is 0.7 percent, and this is without winter measurements. An estimate of the annual average for the buoy, assuming the winter percent occurrence is equal to the fall's, is 0.85 percent, indicating that waves larger than 3.5 m may occur almost four times more frequently than predicted by the hindcast. The above assumption is not conservative, since both WIS and measurements at Burns Harbor indicate significantly larger waves occur in the winter than the fall.

The presence of lake ice at Station 45007, or between Station 45007 and Burns Harbor, would reduce the estimate of exceedance probability by precluding or attenuating wave formation. WIS provides projections of the median ice concentrations on the lake for 2-week intervals over the winter. The median value is used because it was "... subjectively determined that the median ice concentration patterns provided the most coherent pattern of the progression of ice-cover formation and decay over the winter season." The results indicate no (median) ice coverage at Station 45007 or Burns Harbor. Historical records of ice occurrence at Burns Harbor have been described in Chapter 3, but the more conservative WIS convention that this did not materially affect wave statistics at this location will be followed.

For the purpose of estimating wave conditions at Burns Harbor, those waves occurring at Station 45007 which do not propagate toward the harbor should be ignored. WIS puts the total percent occurrence exceeding 3.5 m from the northerly sectors at 0.145, or roughly two thirds of the total number of events. If the same directional distribution is applied to the NDBC (nondirectional) results, the percent occurrence is reduced to 0.56.

The implication of these figures is that the measured conditions at Station 45007 exceed 3.5 m (a condition that may induce damage on the breakwater) two to three times more often than the WIS hindcast predicts. Though the longer period of the hindcast record makes it more valid for climatic projections, the possibility that it underestimates wave conditions needed further testing.

When comparing a discontinuous, measured data set to a continuous synthetic data set, an extremal analysis is preferable to the frequency distribution analysis described above. The ratio of the number of high storm waves measured to all waves measured (the frequency of occurrence) can be affected by the sampling rate, since peak conditions may be of short duration compared to the interval between records. An extremal analysis only considers the distribution of extreme waves over a selected threshold, so their ratio to all waves measured is immaterial. Frequency of occurrence values inherently include all of the available wave measurements, including very calm conditions. In extremal analysis, low energy conditions which do not pose a threat to the structure

are not included; rather, the highest conditions observed over a typical storm duration (i.e., 1 day), are associated with the probability of recurrence of that intensity event.

In addition to the WIS analysis, two extremal analyses were conducted in the course of the study. The first (provided by a consultant, Dr. Michael Andrews; see Volume II, Chapter 3) utilized hindcast data from WIS, and measured data from NDBC Station 45007 and wave gages in Burns Harbor, in a composite extremal analysis to produce revised statistics at WIS Station 62. When WIS Station 64 data between 1985 and 1987 were compared with the NDBC data, it was concluded that 1987 WIS data were biased downward, and the data set would be improved if it were replaced with the NDBC data. An empirical relation developed between WIS Stations 64 and 62 near Burns Harbor was used to transform the 1987 NDBC buoy data to the WIS Station 62. Finally, the Burns Harbor measured data (from wave gages) was substituted into the revised data set whenever it exceeded the WIS Station 62 data. This data set is considered conservative, since it always selects the larger of the available information. A Type 1 extremal model was fitted to the revised WIS Station 62 data and the composite data set consisting of the WIS Station 62 data, the adjusted NDBC data, and one Burns Harbor wave gage measurement.

The second analysis, conducted by NCC, utilized the storm wave history in the stability analysis (Volume II, Chapter 5). In this approach (described earlier in Chapter 4 of this volume), a subset of WIS storm data from Station 62 was compared with measurements from the Burns Harbor wave gage. The conclusion was that the hindcast wave periods were biased downward by 2 sec. After applying the 2-sec adjustment to the WIS storm data set, wave heights from the storm events were transformed into the water depth at the breakwater using Texel, Marsen, and Arsloe (TMA) wave transformation theory. This transformed storm data set was then converted from energy-based wave height to significant wave height. A Weibull distribution model was fitted to the significant wave height transformed data set.

Both of these approaches have their advantages and shortfalls. The approach of the first analysis is more direct and conservative, but does not account for Station 62 shoaling conditions (which are modeled as deepwater waves) to the structure, even though measurements from Burns Harbor were available to develop a third empirical adjustment.

The approach of the second analysis is less direct, utilizing the output of one model (WIS) to feed the input of a second model (TMA). However, it provides for the water depth at the structure, and shows impressive agreement with selected measurements. The second analyis includes a simplistic and significant empirical correction for WIS periods, yet accepts the heights as adequate input, even though it is unlikely the WIS model would produce an error in one without an error in the other.

Ultimately, these three sets of statistics differ little from each other, and markedly from the statistics used in the original design. Table 8 summarizes the results for four return periods. All wave height values have been rounded to the nearest whole foot, because individual measured wave heights have uncertainties of at least 0.5 ft (Coastal Engineering Research Center 1995), and extending extremal statistics beyond the confidence of individual records would be misleading.

Table 8 Return Period and Associated Wave Heights								
Return Period	Original		WIS		MCCP		NCC	
Years	(m)	(ft)	(m)	(ft)	(m)	(ft)	(m)	(ft)
5	3.1	10	4.9	16	4.6	15	4.6	15
10	3.3	11	5.2	17	4.9	16	4.9	16
20	3.7	12	5.5	18	5.5	18	4.9	16
50	4.0	13	5.8	19	5.8	19	5.2	17

Water level extremal analysis. Either an extremal analysis or a frequency distribution analysis is suitable for the measured water level record, because the time series is nearly continuous, and over-sampled relative to the rate of change of the lake levels. Figure 36 (from the extremal analysis) plots water levels versus return interval for the Calumet Harbor gages as taken from a study conducted by the Detroit District.
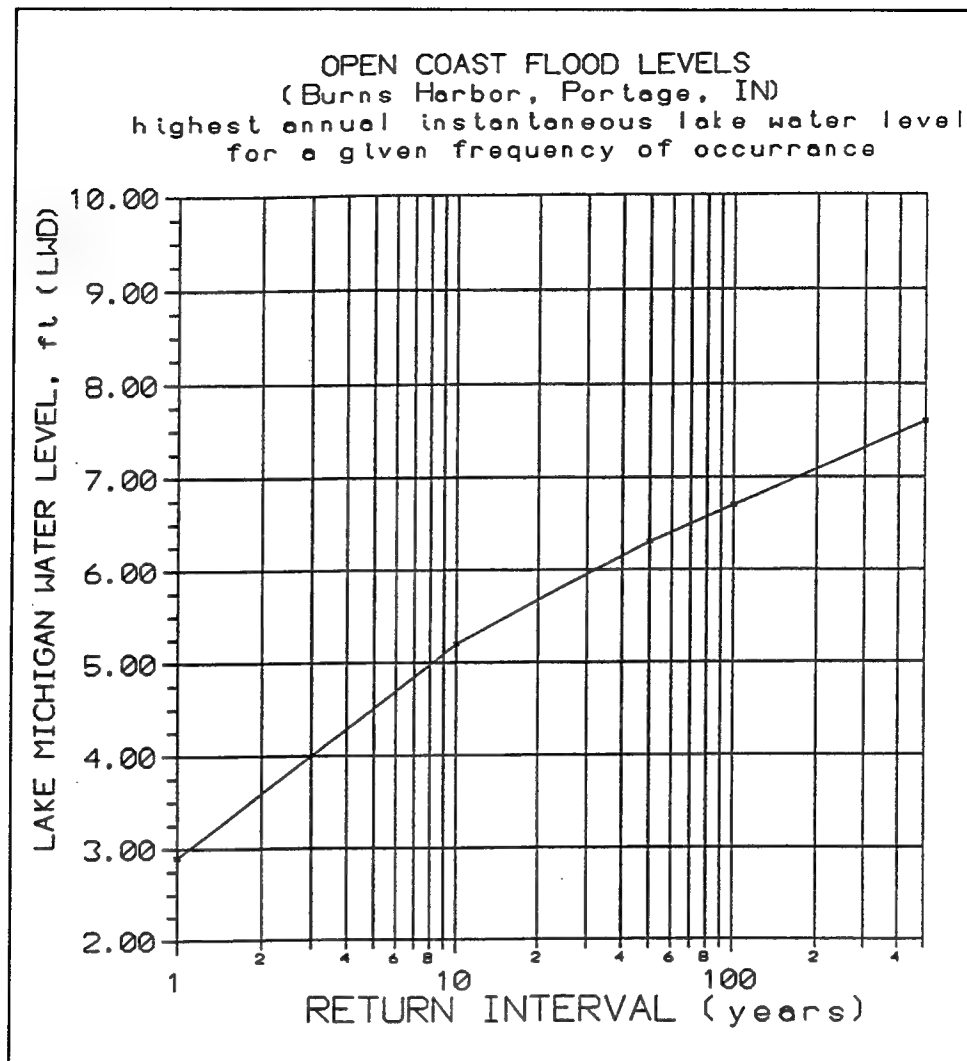
Figure 36. Return interval versus water level

6 Discussion and Conclusions

Conclusions are provided as answers to the most important questions the study attempted to address (Table 3). As is often the case in research, answers are qualified by uncertainty or conditions, or, even worse, raise other questions. For readers with a need for succinct answers, nearly all temptation to equivocate was suppressed and the best, one-word answers provided in Table 9. More detailed answers follow.

Project Attainment

Did the breakwater meet functional requirements for harbor operations? General design practice calls for waves at berthing areas to be less than 1 ft. The current operational criteria at Burns Harbor is a wave height threshold of 3 ft. However, the design documents contain no record of a threshold value or a predicted frequency of exceedance for a threshold. Given the approximately 30-percent transmission coefficient of the breakwater, the 1-ft criteria can be expected to be exceeded routinely, even with the original climatic analysis from the design process. The 3-ft threshold will be exceeded when incident waves exceed 10 ft, which will likely occur several times every year, so one answer is that the project fails to meet operational requirements each year. This was the case since the port was constructed, and is not the result of any degradation of the breakwater's condition or performance.

Since construction of the harbor, the total cargo tonnage transitting the harbor has gradually increased, but with significant interannual variation. In spite of recurring problems and damage, it can be argued that operations have continued, so the harbor is functioning, though not to the satisfaction of users. Inconsistency results from lack of a definitive criteria for describing functionality. A simplistic criteria based on wave height alone fails to consider other factors such as the size of the vessels, mooring practices, and particularly wave period. Motion of moored ships is not linear; the response amplitude can be several orders of magnitude less than, or greater than, the forcing wave height (McGehee 1991b).

Table 9
Monitoring Plan Results

1. Assess Project Attainment	
* Has the project met functional requirements re: harbor operations? navigation?	??? YES

2. Evaluate Design Procedures	
* Was the selection of the design wave appropriate for: overtopping? armor stability?	NO NO
* Was the design water level appropriate?	NO
* Was deposition adequately predicted?	YES
* Did the 3-D model correctly predict waves/ship response at: harbor entrance? berths?	YES NO
* Did the 3-D model correctly predict seiching in the harbor?	YES
* Did the 2-D model correctly predict: wave transmission? stone stability?	NO NO
* Did the foundation perform as predicted?	NO
* Were the effects of ice predicted on: the wave conditions? the structure?	NO ???
* Did the armor units endure weathering as expected?	???

3. Evaluate Construction Methods	
* Was the structure constructed as designed re: foundation? core/bedding stone? sublayers? armor layer?	NO NO YES YES
* Were construction practices appropriate?	NO

4. Evaluate O&M Methods	
* Has maintenance been greater than predicted?	YES
* Have O&M practices been effective at: repairing damage? reducing damage? reducing transmission?	??? NO NO

A more complete description of acceptable operational limits would consider the spectral response of the vessel/mooring system to the harbor's forcing spectrum, and operational thresholds for loading and offloading different types of ships. Finally, the probability of exceedance for various thresholds would be calculated based on the incident climate, so that the costs of structural solutions that affect the harbor response could be compared with the facility downtime. If this type of analysis were conducted for various berths, a more definitive answer to functional performance could be supplied.

Another issue to consider is the different operational requirements for each user. The Cargill facility handles bulk products carried by barge. Barges are more sensitive than large vessels like ore carriers to relatively short, gravity wave energy (periods less than 30 sec). While both types of vessels respond to infragravity (periods between 30 sec and about 5 min) harbor seiching, the dominant rotational modes of bulk carriers tend to be in these lower frequencies, and those of barges in the higher, gravity-wave band. Thus, barge operations have a higher tolerance to seiching (which seldom occurs), but reduced tolerance to short period waves (which occur routinely). The qualified answer to this question is that the harbor meets functional requirements for large vessels, but cannot be considered a year-round port for barge traffic.

The design process that predicted vessel response in the harbor was conducted for a bulk carrier. It is appropriate to point out that the problems experienced by barges are due as much to changes in the functional requirements of the port's customers since its design than any deficiency in predictions of the breakwater's performance.

Did the project meet functional requirements for navigation purposes?
Navigation features of a harbor include the entrance channel, the access channel, turning basin(s), and moorage areas. Because the breakwater was placed in water depths beyond the 30-ft controlling depth for the entrance, no entrance channel was required at Burns Harbor. Also, no access channel was required because the harbor depth up to the berthing areas exceeded the project authorized depth of 28 ft. According to general design criteria, the size of the harbor permits adequate turning area just inside the entrance for vessels less than about 500 ft in length, provided adjacent berths are not occupied.

Design of the harbor entrance configuration resulted from the 3-D physical model study. It was intended to provide adequate navigation clearance while reducing wave energy propagation through the entrance. There have been no incidents of vessel damage due to collision or groundings while transitting the entrance. Pilots and masters of large vessels have described problems in negotiating the port-side turn into the Bethlehem Steel facilities. If sufficient headway is maintained for steerage, it is difficult to decelerate and turn into the east arm of the harbor. The anchor-drag turning method developed by ship operators to cope with this situation appears to be adequate to prevent major problems.

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A third navigational consideration in the design was to avoid problems due to the impact of reflected waves on vessels in front of the breakwater. Historical review did not reveal any such incidents.

Design Procedures

Was the selection of the design wave appropriate ... The process of selecting the design wave, as revealed in the minutes of the two design conferences, can be described as a negotiation. With no measured wave data available for Lake Michigan, a hindcast based on available wind data was used. The wind speed used, 35 mph, was expected to be exceeded 0.031 percent, or for several hours each year, according to the wind statistics. It is not clear how this value was expected to relate to any particular return period, though other documents refer to the design event having a 40-year return period. There were two opinions on the calculation of the fetch distance, and the final selection was simply a compromise. The result was an 11-ft design wave for overtopping, and a 13-ft design wave for stability calculations.

... for overtopping? Establishing a specific threshold for overtopping reveals a view of the transmission processes more suitable to impermeable structures. Because of this structure's porosity, the initiation of overtopping is much less relevant. Since the criteria for setting an overtopping design wave (and the resultant crest height) was to avoid interference with harbor operations, and operations are routinely impacted even before overtopping occurs, the selection of any overtopping wave height was inappropriate. A more suitable approach is selection of an acceptable frequency of exceedence of an operational threshold inside the harbor, and designing the structure's total transmittance to an associated operational design wave outside the harbor with the appropriate return interval.

... for armor stability? Extremal analysis has shown the 13-ft stability design wave to be the 2-year wave. Since the original wave climate was severely underestimated, this parameter is clearly inappropriate.

Was the design water level appropriate? Extremal analysis of the lake water levels has shown the +4.0-ft LWD design water level to have a return period of 3 years. The 30-year design water level would be 6 ft.

Was deposition adequately predicted? The 3-D model study estimated the net longshore transport rate at 27,000 cu yd/year to the west, all of which would be trapped by the landfill constructed at the eastern side of the harbor. Deposition of fine material was expected just inside the north breakwater head. The report concluded that shoaling in the entrance channel by normal longshore transport "... is of no immediate concern." There has been no dredging of the harbor to date, though maintenance dredging is scheduled for 1995. Thirty years can certainly qualify as not "immediate," so the shoaling potential of the entrance was correctly characterized.

Did the 3-D model correctly predict wave conditions and ship response at the harbor entrance? Though wave conditions modeled were lower than design waves developed in this study, the harbor entrance configuration recommended by the study has performed well from a navigation viewpoint. Vessels have not reported problems with reflected wave energy at the entrance. The curved wave absorber is effective in mitigating the difficulties in negotiating the turns required to transit the entrance.

Did the 3-D model correctly predict wave conditions and ship response at the harbor berths? Even taking the reduced incident wave heights into account by normalizing the harbor waves by the incident wave height, the 3-D model greatly underpredicted the wave heights throughout the harbor because it used an impermeable breakwater and only accounted for wave energy coming through the entrance. The actual breakwater is porous and most of the wave energy in the harbor comes through this structure. Modeling transmission through porous structures in 3-D models at these scales would be challenging today, and was probably not feasible at the time of the study.

The ship model tested one type of vessel at several locations. It is assumed the model was unmoored. Measured horizontal motions ranged from "unmeasurable," to a maximum of 9 ft at periods between 110 and 120 sec. The model study concluded the motions of ships in the port would not be a problem of significant concern. Because no specific prototype data were collected on the responses of large ships, a quantitative assessment of the predictions is not possible. However, only one reported instance of damage to a large vessel has been reported, so the conclusion seems to have been verified for large vessels.

On the other hand, there have been numerous instances of damage to moored barges and smaller vessels moored at the grain dock. Wave conditions at the grain dock were not predicted in the design process since it was added after the model test, nor were tests made of moored barges.

Did the 3-D model correctly predict seiching in the harbor? The 3-D model predicted that seiching would be of little concern because of the low incident energy in the lower frequency band of the harbor oscillation modes. Examination of the wave spectra verifies that no discernible energy was measured at periods greater than about 20 sec in either the lakeside or harborside wave gages. Video of vessel response verifies that dominant motions are in the wind-wave band for the lake; i.e., 12 sec and shorter.

Did the 2-D model correctly predict wave transmission? The 2-D model study did not specify in detail how incident wave heights were obtained. If the incident wave height measurements in the model did not include reflected wave energy, and if the wave heights increased behind the measurement location in the prototype in a fashion similar to the model, then the model study underestimated the measured transmission of the <u>existing</u> structure by a factor of about 1/2 for smaller waves, and by about 1/4 for the larger waves.

The existing structure has undergone some evolution from the design tested in the laboratory. Maintenance records show that sufficient repair armor has been placed over the years to effectively turn the breakwater into a three-armor-layer structure. The crest elevation is still near +14 ft, but the core is likely lower than designed because settlement has lowered the core elevation (see discussion of foundation, below). The breakwater's total transmittance is sensitive to the height of the core. The existing structure has a more porous cross section than was modeled, in spite of the additional armor, but there is insufficient data to quantify the effect on transmittance.

Did the 2-D model study correctly predict stone stability? The design wave for the test structure was 13 ft, and there is no clear evidence that waves less than 13 ft caused damage to the breakwater. However, the 2-D model also predicted the structure would be stable for 15- to 18-ft waves. Stable, as defined for the model study, means less than 2.5 percent damages. No damage was predicted on the harborside of the structure. The structure has experienced waves within this range, and damage on both sides of the structure has exceeded that criteria in many locations. Though the wave damage resulted from waves that exceeded the design storm, the model overpredicted the armor stability.

Did the foundation perform as predicted? Excavation and backfill of the foundation were undertaken because the untreated lakebed was predicted to settle about 2.5 ft. No record of the amount this treatment was expected to reduce settlement was found, so this question cannot be addressed for the design phase. Settlement of the treated foundation was predicted under the MCCP study by two different entities. NCC predictions ranged from 0.8 to 1.9 ft along the crest. When weighted by segment length, the average is 1.3 ft. GL predictions for the "as-designed" case ranged from 1.2 to 2.0 ft, with an average of 1.5 ft. GL predictions for the hypothetical case of "remolded" clay in the foundation increased the weighted average by 0.4 ft. If rate of consolidation is included, the predicted settlement by the late 1980's is a little under 1.0 ft, or equivalently, about 40 kt of "lost" armor.

Attempts to measure actual settlement for comparison to these values were severely hampered by the difficulty of calculating volume changes from the sparse historical survey data. The cross section resulting from discrete sampling of the highly irregular cut-stone armor layer is particularly sensitive to the sampling density. The resulting section for any particular station is virtually unreproducible the next day, let alone the next decade. Inconsistencies in horizontal datum control prevented definition of a single structure center line. Finally, documentation of repair activities was insufficient to allow distribution of maintenance stone along the section.

Nevertheless, by applying statistical tools to determine when calculated changes were significant, and by assuming the errors were unbiased and would tend to cancel in an integrated analysis, a reasonable order of magnitude estimate of 100 kt of "missing" stone was determined, representing an average settlement of about 2 ft using the inverse of the equivalent settlement

definition discussed in the "Structural Stability Analysis" section. The equivalent settlement is similar to the upper range of the predicted values, and is very near what was predicted for the untreated foundation.

Were the effects of ice on wave conditions predicted? No. The design studies assumed ice would prevent the formation of waves between December and March.

Were the effects of ice on the structure predicted? Although the presence of ice was assumed in the design phase, its potential impact on the structure was not discussed. There has been subsequent speculation that ice has played a role in armor loss, and it is conceivable that sufficiently thick fast ice under wind or current loading could contribute to unanticipated foundation settlement. However, there are no data to support either hypothesis, and no prediction of the effects would be available to compare to data on the effects of ice, if they existed.

Did the armor units endure weathering as expected? There was no documented allowance for armor breakage or weathering in the design process, so it is assumed that significant weathering of the limestone armor was not anticipated. Photographs of individual armor units fracturing in place document that weathering affected some stones, but not in sufficient numbers to affect the overall structural integrity.

Construction Practices

Was the foundation constructed as designed? There is evidence that the foundation was not constructed as designed. The clay deposition piles lakeward of the structure and the excavation schedule make it obvious that <u>some</u> clay must have been washed back in the trench over the winter, but there is no way of estimating the quantity. The presence of the large sand piles on the lakebed raises the question of whether it is sand intended to be under the structure, but the question cannot be answered definitely. Finally, calculations of settlement with portions of the sand backfill replaced by "remolded" clay are close to equivalent settlement estimates based on stone loss. The answer to this question is "probably not."

Was the corestone placed as designed? The SSS survey, verified by diver inspections, showed that the corestone projects irregularly or not at all on the lakeside. There is no visible corestone on the harborside, even near the entrance where deposition has not obscured the toe. Maintenance activities could have covered corestone at the toe, but the as-built drawings show the corestone generally extending only to the toe, and not projecting beyond it at any stations, particularly on the harborside.

Evidence indicates the corestone layer was not constructed as designed. It is not conclusive that there is insufficient corestone beneath the structure, or that

the failure of the corestone to project beyond the toe at all stations affected the stability of the structure.

Were the sublayers constructed as designed? No practical method was devised to answer this question independently from examination of the as-built surveys. These surveys indicate the sublayer material was properly placed, so the answer is "probably yes."

Was the armor layer constructed as designed? The as-built drawings show a smooth cross section very close to the design template at all stations. Photographs during construction show the visible portion of the breakwater exhibits the expected irregularity of the profile. Apparently, some smoothing of the as-built surveys was employed to represent the average cross section. The existing slopes have armor units extending to the lakebed, whereas the design calls for the armor to terminate at -27 ft and -13 ft on the lake and harborside, respectively. There has been sufficient maintenance over the years to account for this additional armor, so there is no indication the armor layer was not originally constructed as designed.

Were construction practices appropriate? The excavation schedule of the foundation that left clay mounds lakeward of the open trench provided an opportunity for an unknown amount of excavated material to re-enter the trench. There is evidence that a significant portion of the sand backfill material for the trench may have been placed lakeward of the structure. Whatever technique was used to position and place the corestone did not produce the planned 15-ft projection beyond the toe of the W/10 stone.

Maintenance Methods

Has the maintenance of the project been greater than anticipated? The structure was expected to be stable for waves exceeding the design wave; all of the required maintenance was unanticipated at the design stage.

Have O&M practices been effective at repairing damage, reducing damage, and reducing wave transmission? The intent of the maintenance was to rebuild the structure to the design template, by replacing stone assumed lost by wave action, and reduce transmission into the harbor. The segments exposed to the most wave damage - 1 through 6 - are currently near or above the design template, so repairs are in general keeping up with damage in these segments. Crest elevation, however, is on the average about 1 ft below the design elevation. Segments 7 and 8 have received the least repairs and are currently deficient in area, so increased maintenance is called for in these segments. The repairs have not resulted in significant damage reduction, but since the design was not modified, that was not the intent.

In some instances on the harborside, armor stone was placed in a more laid-up style to seal the structure to wave action. Though specific wave measurements were not obtained, the size of these laid-up sections was insignificant in

relation to the overall structure, and wave transmission was not affected. This attempt at improving performance was ineffective, and may have resulted in a less stable backslope. If harborside damage is primarily due to <u>overtopping</u> waves, the laid-up sections should be more stable. If harborside damage was caused by waves coming <u>through</u> the structure and "blowing out" harborside armor, reduced porosity and a steeper slope would be less stable. Without specific studies on these modes of failure, the effectiveness of that placement technique remains questionable.

Inspection surveys of repaired sections show a tendency for the structure to have bowed out at the waterline, resulting in a steeper slope, particularly on the harborside. This is consistent with the fact that the average cross-section area has been maintained, while the crest elevations are low. This may have resulted from O&M practices that were attempting to tighten the structure, and thus reduce transmittance, while accomplishing needed repairs. Care should be taken in future maintenance activities to maintain design slopes.

Problem Solving

In addition, six "bottom-line" questions (Table 3) of particular impact to the operation and management of the project in the future were addressed.

Did the breakwater experience a failure? The answer to this question would be straightforward if it were definitely no (no discernible damage) or definitely yes (catastrophic collapse). Rubble-mound designs are particularly suited for coastal structures because they can tolerate some damage without failing. The Burns Harbor breakwater has experienced considerable damage over its life, but no single storm has caused complete loss of a section to below the waterline, such as experienced by the King Harbor, CA, breakwater after the February 1989 storm. In fact, associating damage with any specific event has been difficult. At what point does unanticipated maintenance become failure?

Determination of failure must take into consideration the meaning of the word itself. For example, in Basco (1992) failure is defined as "when energy levels in storms *below* the design levels cause loss of structural integrity and/or functional performance, then a failure in the design has resulted." Most coastal design is probability-based due to the random nature of the loading. If conditions exceed the design wave, the resulting wave damage should not be considered a failure of the <u>design</u> of the structure. There is no evidence the structure suffered significant armor damage under conditions less than the design wave, so the design of the cross section was not a failure by that definition.

Likewise, damage that does not result in loss of structural integrity is not a failure of the structure. The Burns Harbor breakwater experienced waves larger than its design condition numerous times, and the structure was

damaged extensively, but in small (relative to structure length) localized sections. Maintenance efforts, though higher than anticipated, were effective in repairing the damage. However, repeated instances of exceedence of the design wave, as happened at Burns Harbor, certainly indicate failure to predict the wave climate and the resulting damages to the structure.

The structure has settled by an amount very near what was predicted if the structure were built on the existing lakebed. Since the foundation treatment failed to reduce that settlement, it was a failure either of design or implementation.

The structure has also failed to functionally perform as predicted in reducing wave energy. Although this could be a result of failure of the modeling technique, the more likely conclusion is that the existing structure is more porous than the design. There is no indication that wave damage alone resulted in significant increase in the transmission coefficient. In fact, there was no significant damage to the armor layer or loss of crest elevation at the section in front of the harbor wave gage during the measurement interval. However, if there is additional armor on the structure, and the structure is not correspondingly larger, then the impermeable portion of the cross section below the armor must be lower, making the overall structure more porous.

How is wave energy entering the harbor? Most wave energy comes through the porous armor layer of the breakwater, as opposed to through the entrance. When waves are sufficiently high, overtopping will also occur and contribute to transmission, but there is no abrupt transition on wave transmittance before and after overtopping.

How can wave energy in the harbor be reduced? Options are discussed below. Design studies would be required to test their suitability.

a. Raise the crest elevation. Modifications to the crest will only affect that portion of the wave energy that is currently overtopping the structure. Though total transmittance appears to increase when overtopping commences, the transmission coefficient on the order of 25 percent for non-overtopping conditions will continue to let significant energy into the harbor, regardless of crest elevation.

The crest could be raised with additional armor or with the addition of a concrete cap, as was suggested in the preliminary design. A cap would probably improve stone stability as well, but a smooth cap may enhance runup and overtopping of the higher waves by providing a smooth surface. A grid-like "rib cap" may be as effective a stabilizer as a solid cap while requiring much less material, and would not enhance runup (Figure 37).

b. Seal the structure. Some options are available to tighten the structure by sealing voids with grout or asphalt. The long-term durability of these options is not well-known. It is possible that the existing armor
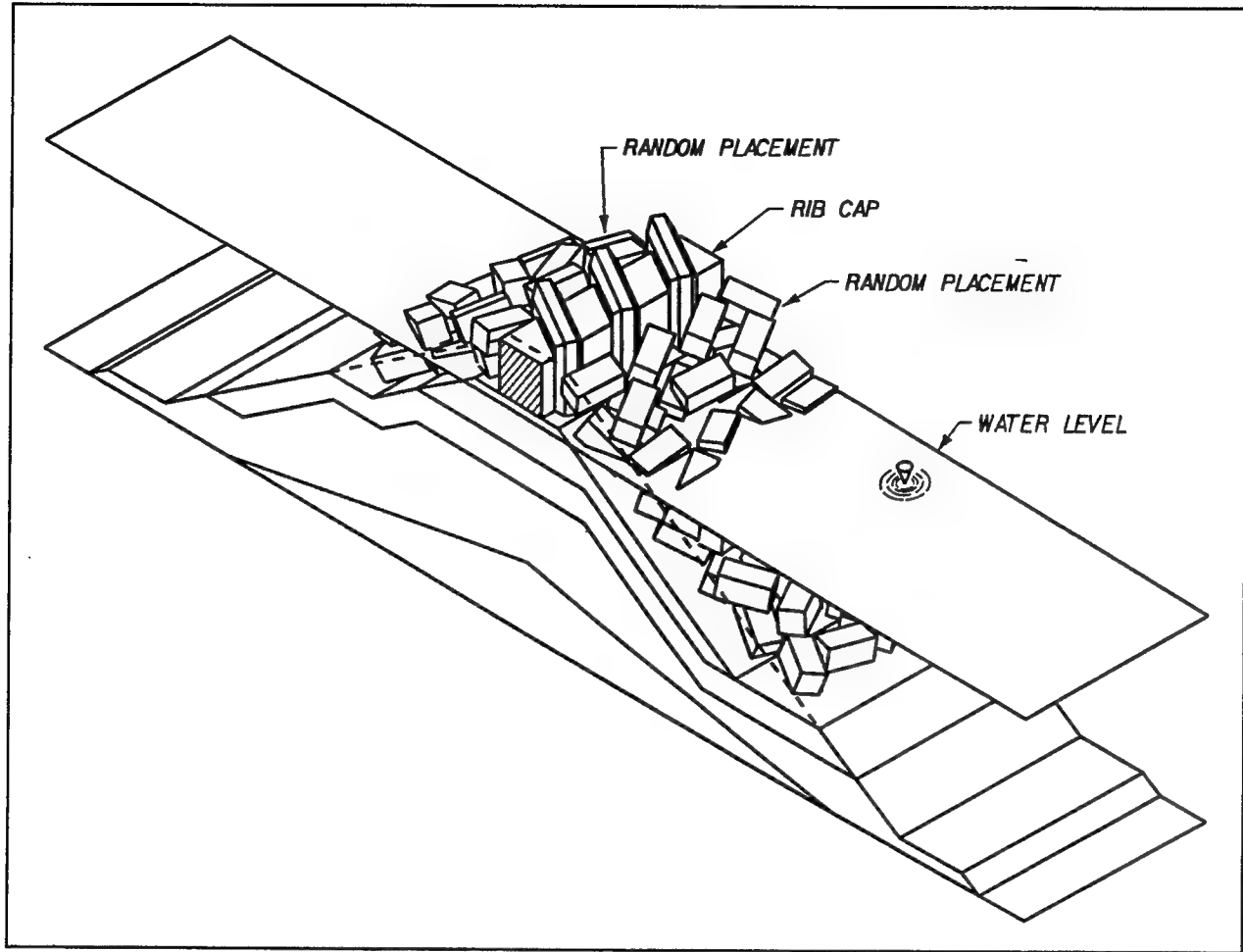
Figure 37. Illustration of rib cap

could be repositioned to achieve lower porosity, but those options are effectively restricted to the subaerial portion of the structure because they cannot be practically accomplished below the waterline.

The effect of reducing porosity on the transmittance and stability of the structure should be carefully considered and tested. A less porous structure will likely experience more runup, so overtopping would increase, possibly increasing total transmittance at larger wave heights. Energy not transmitted through the structure will be either reflected or absorbed, possibly with detrimental effects on the structure stability.

c. Add more stone to the structure slope. Additional layers of stone on either the lakeside or harborside slope would reduce transmission. Given the porous nature of layers made with the cut-stone armor units, it will take significant amounts of this stone to produce noticeable decreases in transmission. Significant damage to both the lake and harborside of the existing structure indicates that any additional stone should be larger and/or placed on a shallower slope. There is little

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available room inside the harbor to permit extension in that direction. Finally, given the strong evidence of settlement, it is not recommended that any additional loads be placed on the foundation without extensive geotechnical investigations (slant borings under the structure, vertical borings through the structure, ground-penetrating radar surveys,etc.) to better evaluate additional potential settlement.

d. Place a protective structure in front of the breakwater. An additional, detached structure in front of the existing breakwater could reduce wave energy transmission. This would avoid questions of the existing structure's foundation reliability. It will likely take a significant structure to effectively reduce transmission. To make a simplistic analogy, it would take another structure of the same size as the existing one to reduce the transmission to 30 percent of what came through the first structure, so that the combined transmission coefficient would be on the order of 10 percent, i.e., the 50-year return wave decreased'to about 2 ft in the harbor.

What is the reason for the frequent and unanticipated maintenance? Two factors combined to cause a need for frequent maintenance of the structure - wave damage to the armor layer, and settlement of the foundation.

Wave damage occurred because of the routine occurrence of waves high enough to damage the structure. The structure was underdesigned for this wave climate. A second factor in the wave damage is the highly variable stability of the rectangular armor units. Interlocking of these units is very sensitive to their placement. While one random arrangement of random shapes, such as quarrystone, behaves very much like another random arrangement, the same is not true for regular shapes. One random arrangement of rectangular shapes may be very stable, but repositioning just a few units may make the whole stack very unstable. The demonstration of this fact is the variable response of the breakwater; some sections were damaged extensively during events that left adjacent sections unchanged.

Failure to predict actual settlement is due to either failure of the prediction tools, or deviance of the existing foundation from the design assumptions. Given the highly variable nature of the sediments under the lakebed, and the strong evidence that the structure was not constructed as designed, the latter option is the more likely.

A third possibility is the combination of both reasons; i.e., because the structure was not built as designed, the wrong prediction tool was used. Predicted settlement was based on consolidation from vertical compression, but failure may be localized at the toe. This hypothetical scenario begins with a foundation consisting at some locations of a thin layer of sand over redeposited clays. Next is a sporadic covering of bedding stone. Finally, additional maintenance stone is placed along the side slopes all the way to the toe where it rests on untreated lakebed. Long waves, particularly those arriving off-normal, liquify the unprotected sand at the lakeside toe causing slope failure of the

armor units above (Tsai 1995). At least one partially buried armor stone was observed on the lakeside to support this hypothesis. This mechanism is conceded by several sources to be responsible for the failure of the breakwater at Sines, Portugal (Smith and Gordon 1983, Silvester and Hsu 1989). The dimensionless analysis technique of Sakai (1992) places the lakebed sediments well within the liquefaction range for typical incident waves. On the harborside, the mud mound could be an indication of a slip failure. Though a slip-plane failure was ruled out during the geotechnical reanalysis, it was based on the <u>designed</u> foundation and didn't consider the possibility of liquefaction.

How can maintenance be reduced? Options are discussed below. However, since no economic analysis was conducted, no implication is intended that any of these would actually result in reduced overall costs. Some of the options will also reduce wave transmittance, but the design of a modification could vary depending upon which objective was emphasized.

a. Add larger stone and/or reduce the angle on the slope(s). In addition to this obvious option, variations involving toe berms or reshaped slopes using smaller stone at very shallow angles and below the waterline should improve stability. The previous caveat regarding a geotechnical investigation would apply.

b. Add a concrete cap to the structure. A properly designed concrete cap would improve stability of the crest. It would not address displacement of armor further down the slope. The potential for increasing damage through increased reflected energy to lower parts of the structure, as mentioned for void-sealing, should be carefully considered.

c. Place a protective structure in front of the breakwater. While reducing the total wave transmission to 10 percent may require another break-water, a significant portion of that reduction may be accomplished by a reef-type structure well below the water level, using relatively small stone. If a protective structure reduced the incident wave energy to 60 percent of the incident height, the 50-year return wave reaching the existing structure would be reduced to 12 ft, or below the original design wave.

What is the prognosis for the present structure? There is no indication that wave damage will decrease. While there is considerably more armor on the present structure than the design, there are no test data available to indicate if multiple layers (more than two) of cut stone armor are more or less stable than two layers. Wave damage will probably continue to occur.

Calculations of rate of soil consolidation predict about 80 percent of settlement has occurred. Though the magnitude of settlement was underestimated, consolidation settlement is not likely to be a major contributor to future damage. If the scenario described above has occurred, failure at the toe should be investigated. This type of failure may be more localized, and the structure may continue to exhibit those effects in the future.

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Lessons Learned

Some guidance of a general nature that is applicable to management of any coastal structure is provided in response to the question: *What lessons were learned about conducting monitoring studies of coastal projects?*

a. A decision to fully monitor a project should be accompanied by a generous committment of time and resources. While geographical bounds can be placed around the project, the technical considerations can include the entire range of subjects known as coastal engineering. The techniques used to obtain many of the measurements required to quantify processes are areas of ongoing research; they may require development, and are not always successful. While low-level monitoring is extremely useful to ascertain the condition of a project, it is not likely to ascertain the causes or the modes of failure.

b. Monitoring a "problem" project involves considerable amounts of forensic engineering to identify modes and causes of damage. The quality of results is directly related to the quality and availability of the documentation of the project history. Each major coastal project should be considered a potential problem from inception, in the sense that detailed histories are maintained. These should include the following:

   (1) Routine inspections made with photographic and/or video documentation of project condition.

   (2) Special inspections conducted as soon as possible after episodic events to determine the extent and pinpoint the time of any damage so it can be related to the conditions that caused it.

   (3) Bathymetric and structure surveys made with common, repeatable vertical and horizontal control. In particular, the emphasis on structural surveys should not just be determining the damage (i.e., the difference between the current profile and the design profile), but the difference between the current profile and earlier profiles, so that the evolution and mode of damage can be traced.

   (4) Riprap breakwater surveys should be conducted in such a manner as to afford complete cross-section coverage. Spot elevations should be taken and recorded along predefined intervals and at surface break points. Strict field quality control should be implemented to avoid "holes" or "data omitted" areas along each cross section. This may be challenging near the waterline during marginal weather conditions. Objective surveying is required if objective analysis is to be performed using such data.

   (5) Most commercial fathometers use a frequency and beam pattern that are a compromise between shallow and deepwater applications. When used on rubble-mound structures, returns from side

lobes of the acoustic beam rebounding from stones higher up the slope may be stronger than the return from directly beneath the transducer. These soundings, biased shallow until away from the structure, produce a swelled profile. Use of a high-frequency, narrow beam transducer will reduce these effects.

It is suggested that future surveys of the Burns Harbor breakwater utilize mechanical sounding methods (lead line and sounding basket or sounding pole) (McGehee 1987).

(6) Improved documentation of maintenance/repair activities that include details such as the placement technique and distribution of stone along a profile, not just the total tonnage placed in a section. Any assumptions or goals of the repair, such as improving interlocking or sealing voids, should be documented. Before and after photographs of repairs should be taken.

c. Monitoring plans, and particularly data collection plans, should be carefully designed and directed toward answering specific, important questions. A data analysis plan should be developed before the data collection plan to ensure the data captured are of sufficient duration, quality, and from the right location to answer these questions.

7 Summary

A monitoring study of Burns Harbor, IN, was conducted to evaluate the design process and identify the causes of complaints of excessive wave energy by harbor users and frequent maintenance requirements. The study included an in-depth historical review, and collection and analysis of data on the breakwater foundation, breakwater stability, and wave characteristics inside and outside the harbor.

The structure was determined to be under-designed, principally due to underestimation of the wave climate in Lake Michigan. An improved hindcast, supplemented with wave data, produced an updated extremal analysis. The original 13-ft design wave was determined to be a 2-year event.

The cut stone armor used in the breakwater exhibited a wider variance in stability than associated with typical rubble-mounds. The result is a highly variable pattern of damage on the structure. The stability of cut stone armor is more sensitive to placement technique than other types of armor. Weathering of the armor resulted in some breakage, but not a significant amount.

The structure may have experienced greater than anticipated settlement, though the difficulty of evaluating historical survey data and variation in settlement along the structure hampers attempts to estimate the actual settlement. Both the original geotechnical design and a subsequent reanalysis predicted average settlement of about 1 ft. However, statistical analysis of the survey data suggests the structure has settled an average of about 2.0 ft. This settlement represents a "loss" of armor stone on the order of 100 kt, roughly equivalent to the amount of repair stone placed on the structure over its life.

A 3-D model study used to plan the harbor resulted in an effective entrance design. This study did not accurately predict wave conditions in the harbor because it assumed all wave energy would enter the harbor through the entrance (impermeable breakwater, no overtopping), and it underestimated the design wave.

A 2-D model study used to design the breakwater cross section underestimated wave transmission, possibly caused by settlement of the structure and subsequent repairs that resulted in a more porous structure. The 2-D model study appeared to predict the stability, within the variability described

above, but did not accurately predict the harborside damage which was approximately equal to the lakeside damage over the life of the structure.

The functional requirements of the project have changed since design due to an increase in barge traffic in the harbor. Most of the user complaints regarding operations can be traced to one facility, the grain dock on the north wharf, which is constructed with a vertical sheet-pile face. Measurements verify that reflection caused wave conditions in front of the dock to be twice the height of waves in the open area of the harbor. This facility was not anticipated at the design phase.

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Appendix A
Index of Documents and Drawings Obtained in Historical Review

1. Documents listed on pages A2-A8 are located at the Coastal and Geo-technical Branch, Engineering Division, U.S. Army Engineer District, Chicago, 1111 North Canal, Chicago, IL 60606.

2. Documents listed on page A9 are located at the U.S. Army Engineer Waterways Experiment Station, Records Management Cener, 3909 Halls Ferry Rd, Vicksburg, MS 39180-6199, ATTN: CEWES-IM-P, Reference Marks File No. 1110-2-14036.

STRUCTURAL DESIGN

Experiments with cellular type steel sheet pile breakwater models, 1933.

Appendix E. Burns Waterway Harbor Design Memorandum north breakwater and west outer bulkhead, preliminary design, March, 1966.

Burns Harbor north breakwater and west outer bulhead cost estimate, March, 1966.

Addendum No. 1 to specifications and contract documents for contract II-A, May, 1966.

Technical report rubblemound breakwater, 1985.

Burns Waterway Harbor specifications and contract documents for north breakwater, west outer bulkhead, and east inner bulkhead contract II-A.

Burns Waterway Harbor specifications and contract documents for work harbor entrance, temporary north and south bulkheads, contract II-A1.

GENERAL DESIGN

Burns Waterway Harbor Design Memorandum, March 1966.

Burns Waterway Harbor specifications and contract documents, 1967.

Burns Waterway specifications, July 1980.

Burns Harbor stone specifications, May 1985.

Appendix G, Burns Harbor Design Memorandum.

Burns Waterway Harbor specifications and contract documents for site grading and temporary access road.

Burns Waterway Harbor specifications and contract documents for harbor dredging, site grading, and inner bulkheads, contract II-B

Burns Waterway Harbor specifications and contract documents for port terminal RR and initial track.

Burns Waterway Harbor specifications and contract documents for port terminal RR and initial track extension, contract IV-B

CONSTRUCTION

Burns Waterway Harbor project notes on current situation, Nov 1965.

Burns Waterway Harbor stage 1 progress report no. 1, Dec 1965.

Burns Waterway Harbor stage 1 progress report no. 2, Feb 1966.

Burns Waterway Harbor stage 1 progress report no. 3, Apr 1966.

Burns Waterway Harbor stage 1 progress report no. 4, Jun 1966.

Burns Waterway Harbor stage 1 progress report no. 5, Aug 1966.

Burns Waterway Harbor stage 1 progress report no. 6, Oct 1966.

Burns Waterway Harbor stage 1 progress report no. 7, Dec 1966.

Burns Waterway Harbor stage 1 progress report no. 8, Feb 1967.

Burns Waterway Harbor stage 1 progress report no. 9, Apr 1967.

Burns Waterway Harbor stage 1 progress report no. 10, Jun 1967.

Burns Waterway Harbor stage 1 progress report no. 11, Aug 1967.

Burns Waterway Harbor stage 1 progress report no. 12, Oct 1967.

Burns Waterway Harbor stage 1 progress report no. 13, Dec 1967.

Burns Waterway Harbor stage 1 progress report no. 14, Feb 1968.

Burns Waterway Harbor stage 1 progress report no. 15, Apr 1968.

Burns Waterway Harbor stage 1 progress report no. 16, Jun 1968.

Burns Waterway Harbor stage 1 progress report no. 17, Aug 1968.

Burns Waterway Harbor stage 1 progress report no. 18, Oct 1968.

Burns Waterway Harbor stage 1 progress report no. 19, Dec 1968.

Burns Harbor, Svedrup and Parcel, memorandums, construction related, 1965-1969.

History of construction

ENVIRONMENTAL

Degree of pollution of bottom sediments in Burns Harbor, July 1970

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Sampling and analysis of west dock extension Burns International Harbor, December 1985.

MCCP RELATIVE TO BURNS HARBOR

Burns Waterway Harbor MCCP, letter proposing a monitoring program at Burns Harbor, May 1984.

MCCP interim report FY86.

MCCP interim report FY86, plates 4-21.

MCCP FY86 report plates 4-24 xerox copies.

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MCCP engineer program, FY86, letter file having excerpts concerning results of surveys and analysis.

MCCP interim report FY87.

MCCP nominations, 1988.

MISCELLANEOUS

Great Lakes harbors study interim report on Burns Waterway Harbor, Indiana, January 1962.

House Document No. 160, Great Lakes harbors study interim report on Burns Waterway Harbor, Indiana, October 1963.

COASTAL DESIGN

Exhibit "A," hydraulic model study, December 1964.

Burns Waterway model study photos and progress report, 1966.

Burns Harbor, NESOC report SN-311, development of shoreline configuration west of Burns Harbor after completion of port and fills west of the west jetty, February 1966.

Appendix D, Burns Waterway Harbor Design Memorandum, hydraulic model study, March 1966.

Burns Waterway Harbor, wave and shoreline studies, calculations and newspaper clippings, 1961-1967.

Burns Harbor model test breakwater design 62-67, letters concerning designs, bill, and vouchers.

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Technical specifications for west jetty, Indiana Lake Port Midwest Steel Division, January 1967.

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Burns Harbor responses, soundings, 1975.

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Burns Harbor responses, field wave measurement program progress report, February 1978.

Burns Harbor responses, digital analysis of wave gage data, March 1978.

Wave climate for Burns Harbor, December 1978.

Coastal Hydraulic Models, SR-5, May 1979.

MCCP Meeting, 15 May 1984, hybrid element modeling of harbor resonance.

Burns stone placement, side-scan sonar survey, June 1985.

Burns Harbor wave gage data 1986.

CERC wave data analysis, Burns Harbor Indiana, December 1985 and January 1986.

GEOTECHNICAL DESIGN

Burns Harbor Design Memorandum log of borings folder, boring logs and test results, May 1960.

Burns Waterway Harbor specifications for offshore soils exploration and preliminary onshore soils exploration, March 1965.

Burns Harbor subsurface condition report, November 1965.

Appendix F, Burns Waterway Harbor Design Memorandum, subsurface conditions, March 1966.

Study of sand fill in Bethlehem Steel Corp. riparian area, May 1966.

Technical report no. 2-766, "Stability of Proposed Breakwater, Burns Waterway Harbor, Indiana, March 1967.

Burns Harbor Bethlehem Steel plant screening folder, preliminary stability analysis east jetty and riparian wall area, Burns Harbor plant, Bethlehem Steel Corp., March 1970.

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REPAIRS AND MAINTENANCE

1980-1983 Burns emergency stone placement.

MCCP miscellaneous information (stone placements), 1985 scope of work.

FY87 MCCP miscellaneous files. MFRR Burns Harbor breakwater damage survey, June 1987.

Burns Waterway Harbor, stone placement information, computer disks, August 1987.

Burns Harbor breakwater stone placement pictures.

Dredging.

Emergency stone, Burns Harbor.

DRAWINGS

Suggested Burns Waterway Harbor. January 1993.

Plan of Improvement and Wave Refraction Study. Drawing No. 60-6-R2. January 1960. 4 sheets.

Land Use. Drawing No. 60-6-R2/1A. January 1962.

Indiana Shore at Lake Michigan in Vicinity of Burns Waterway Harbor. Drawing No. 60-6-R2/1B. July 1963.

Burns Ditch Waterway-Harbor Plan Proposed by Save the Dunes Council Engineering Committee. Drawing No. 60-6-R2/1B. July 1963.

Burns Waterway Harbor Site. Exhibit "B". Proposed Locations for Breakwaters and Bulkheads. Drawing No. 2397-402. May 1968.

Dock and Riparian Area Facilities, Burns Harbor. Drawing No. P47c. November 1969.

Indiana Port Commission, Burns Waterway Harbor. Sheet 1 of 9. January 1971.

Wave Height Measurement Location Plan. Drawing No. 187602. November 1974.

Layout of Riprap Breakwater. Drawing No. BWH-1. June 1975.

Real Estate Burns Waterway Harbor, Indiana. Drawing No. NCD-2-0022. May 1965.

Control Map, Burns Harbor. Drawing No. 60-6-S1/1. June 1975.
Location MCCP Files, MCCP Burns Breakwater Sounding Program Plan E.

REPAIRS

North Breakwater Stone Placement. Drawing No. 60-6-B6/1. 6 sheets. September 1985.

Stone Placement 22+77-30+52. "As-Builts." October-November 1985.

Berth 15 Storm Damage Dredging, Indiana Port Commission. Drawing No. Contract 11-5-3. 4+ sheets.

CONSTRUCTION

North Breakwater and West Outer Bulkhead, Removal of Soft Clay, As-Built, Cross Sections, Sta. 85+00-145-95. 31 sheets. Drawing No. 2397-370 through 400. February 1968.

North Breakwater and West Outer Bulkhead, As-Built, Cross Sections, Sta. 85+00-145+95. 31 sheets. Drawing Nos. 2397-232 through 240D. January 1969.

SOUNDINGS

Proposed Burns Waterway-Examination Soundings. Drawing No. 60-6-E2. September 1959.

Indiana Port Commission Harbor Dredging. 4 sheets. October 1968.

1973 Examination Soundings, Burns Waterway Harbor. Drawing No. 60-6-Ex. April 1973.

Burns Waterway Harbor, Indiana. June 1973.

Burns Waterway Harbor, Indiana. 1975 Riprap Soundings, North Breakwater Vicinity and Harbor Plans. 5 sheets. Drawing No. 60-6-R8/1. June 1975.

1976 Examination Soundings. Burns Waterway Harbor. Drawing No. 60-6-E2/3. August 1976.

1978 Examination Soundings, Burns Waterway Harbor. Drawing No. 60-6-E2/4. April 1978.

1979 Examination Soundings, Burns Waterway Harbor. Drawing No. 60-6-E2/5. August 1979.

Condition of Channel. May 1982. Drawing No. 60-6-E2/6. July 1982.

Condition of Channel. June 1983. Drawing No. 60-6-E2/7. July 1983.

Condition of Channel. August-September 1984. Drawing No. 60-6-E2/8. October 1984.

Structure Survey. August-September 1985. Sheets 3, 4, 5.

Condition of Channel. October 1986. Drawing No. 60-6-E2. October 1986.

BOX 1 OF 4: WES-GENERATED DOCUMENTS (1965-1996)

Reports
Historical Records
Memos
Project Files
Analyses

BOX 2 OF 4: NCC-GENERATED DOCUMENTS (1965-1996)

Reports
Historical Records
Memos
Photographs

BOX 3 OF 4: DATA (1965-1996)

Wave (printed)
Bathymetric Surveys
Cross-section Surveys
Subbottom Sonar Surveys
Side-scan Sonar Surveys

BOX 4 OF 4: MCCP BURNS HARBOR (1965-1996)

Station by station data compilation

Appendix B
Dive Inspection Reports

DISPOSITION FORM

For use of this form, see AR 340-15, the proponent agency is TAGCEN.

REFERENCE OR OFFICE SYMBOL	SUBJECT
CENCE-CO-DE (1130e)	BURNS DITCH WATERWAY

TO	FROM	DATE	CMT 1
THRU: Chief, Detroit Area Office Chief, C-O Division TO: Chief, O&M Branch	Chief, O&M Section Detroit Area Office	21 July 1988 Fitzhugh/dbl/66855	

1. REFERENCE: 1 June 1988 Memorandum from Chief, Engineering Division, Chicago District to Commander, Detroit District, subject "Support Assistance in the Monitoring of Completed Coastal Project (MCCP) Program at Burns Harbor, In.

2. PURPOSE: To inspect designated areas of the South side of the main breakwater substructure.

3. DATE and PLACE: 5 - 6 July 1988 at Burns Harbor Waterway, Ill.

4. PARTICIPANTS:
Diving Supervisor	–	Mr. George Fitzhugh
Assistant	–	Mr. Arnold Rybak
Tethered Diver	–	Mr. James Bumford
Tethered Diver	–	Mr. Darrol Shedenhelm
Tethered Diver	–	Mr. Ken Zmikly
Chicago District	–	Ms. Heidi Pfeiffer
Chicago District	–	Mr. Charles Johnson
Chicago District	–	Mr. Rod Moritz

5. SIGNIFICANT ELEMENTS OF THE REPORT: Divers inspected the areas designated by the Chicago District personnel and gave an on-the-spot verbal report to them. Our observations are noted in the narrative.

6. NARRATIVE: Divers started at station 0+00 on the East end of the break water and proceded Westward. The results of the inspection are as follows:

   a. Sta. 0+00 - Slope ends at 115' South of the breakwater. No large voids in the armor stone were noticed. It appeared as if the armor stone had rolled down to the slope toe. The slope is uniform except where armor stone had rolled down. Some mattress stone was noticed.

   b. Sta. 0+50 - Slope ends at 80' South of the breakwater. Armor stone ends at 75' South. Slope appears normal to 70' South and then drops vertically 10 feet down to mattress stone with scattered armor stone in area. There is also fractured armor stone on slope.

   c. Sta. 1+00 - Slope ends at 95' South of breakwater. The armor stone appears to be laying haphazardly down the slope to 55' South where there is a 15 foot verticle drop to mattress stone. There was no fractured armor stone noticed in this area.

DA FORM 2496, 1 FEB 62   REPLACES DD FORM 96, WHICH IS OBSOLETE.   1   ☆U.S.GPO:1979-0-310-981/8129

d. Sta. 1+50 – Slope ends at 95' South of breakwater. The slope appears to be normal and covered with armor stone. No mattress stone was noticed.

e. Sta. 2+00 – Slope ends at 95' South of breakwater. The armor stone appears to be laying haphazardly down the slope to 55' South where there is a 10 – 12 foot verticle drop to mattress stone. Although there was no large voids noticed, some of the armor stone has rolled down to the toe of slope. Some armor stone is laying approximately 20 feet south of the toe. There was no fractured armor stone noticed.

f. Sta. 2+50 – Slope ends at 70' South of breakwater. There is a 30' x 10' x 15' deep void at the face otherwise the slope appears to be normal and covered with armor stone. There is some armor stone 5' South of toe of slope and mattress stone noticed in area.

g. Sta. 3+00 – Slope ends at 85' South of breakwater. There is a 10' x 10' x 10' deep void at the face otherwise the slope is normal to 40' South where there is a 10' verticle drop and the armor stone is laying haphazardly to 5' past the toe. Some mattress stone was noticed in this area.

h. Sta. 3+50 – Slope ends at 95' South of breakwater. The slope and armor stone is normal to 45' South where there is a 13 foot verticle drop then the slope continues to the toe. Some mattress stone was noticed in this area.

i. Sta. 4+00 – Slope ends at 90' South of breakwater. The slope and armor stone is normal to 15' South where 1 ton stone begins and continues to the toe. Slope appears to be uniform. There were no large voids or mattress stone noticed in the area.

j. Sta. 4+30 – Slope ends at 100' South of breakwater. The slope and armor stone appears to be normal with no large voids although there is some loose armor stone laying at the toe. Some mattress stone was noticed in the area.

k. Sta. 4+50 – Slope ends at 90' South of breakwater. The slope and armor stone is normal to 15' South where 1 ton stone begins and continues to the toe. Slope appears to be uniform although at 20' South there is some fractured armor stone. There were no large voids. Some mattress stone noticed in the area.

l. Sta. 4+80 – Slope ends at 90' South of breakwater. The slope and armor stone is normal to 15' South where 1 ton stone begins and continues to the toe. Slope appears to be uniform. No fractured armor stone noticed. There were no large voids although there as some armor stone scattered out past the toe.

2

m. Sta. 5+00 – Slope ends at 90' South of breakwater.   The slope and armor stone is normal to 15' South where 1 ton stone begins and continues to the toe. Slope appears to be uniform. No fractured armor stone noticed. There were no large voids although there as some armor stone scattered out past the toe.

n. Sta. 5+50 – Slope ends at 105' South of breakwater. The slope and armor stone is at a shallow angle to 55 feet South (water over armor stone is 6 feet deep at 31' South) where the slope drops vertically 15 feet and then continues to the toe on a steeper than normal angle.   There are some small voids in the armor stone. No mattress stone noticed.

o. Sta. 6+00 – Slope ends at 90' South of breakwater.   The slope and armor stone is at a shallow angle to 35 South (water over armor stone is 5 feet deep at 35 South) where the slope continues to 65' South where the slope drops vertically 15 feet and then continues to the toe. There are some small voids in the armor stone. No mattress stone noticed.

p. Sta. 6+25 – Same as above.

q. Sta. 6+50 – Same as above.

r. Sta. 7+00 – Slope ends at 100' South of breakwater. The slope and armor stone is normal to 60' South where there is a 8 foot verticle drop and then continues to the toe. The armor stone appears to placed haphazardly all the way to the toe. Some voids were noticed.

s. Sta. 8+00 – Slope ends at 80' South of breakwater. Except for a 10' bridged void at the water line, the slope and armor stone is normal to 40' South where 1 ton stone begins and continues to the toe with a uniform slope. No fractured armor stone noticed.

t. Sta. 25+80 – Slope ends at 80' South of breakwater. The slope is gradual and appears asbuilt. No Fractures or voids in the armor stone noticed.

u. Sta. 26+00 – Slope ends at 90' South of breakwater. The slope and armor stone is normal to 45' South where there is a 10 foot verticle drop and then continues to the toe.

v. Sta. 36+00 – Slope ends at 75' South of breakwater. The slope and armor stone is normal to 35' South where some armor stone has rolled down to 45' South then drops vertically 12 feet and continues to the toe. Some fracturing of the armor stone was noticed.

w. Sta. 40+00 – Slope ends at 80' South of breakwater. The slope and armor stone is normal to 40' South where there is one piece of armor stone laying misplaced on the slope appearing to be asbuilt. There were no fractured stone noticed. The bottom was sandy in this area as opposed to being mud and silt elsewhere.

3

7. <u>CONCLUSION:</u> The distance to the toe of slope was measured from the center line of the breakwater. No underwater photographs were developed due to a malfunction in our camera.

Thomas E. Ottenbaker
Agency Diving Coordinator
Chief, O & M Section
Detroit Area Office

cc: CENCC-ED-G
    Ms. Heidi Pfeiffer

4

CEWES-CD-P   July 7, 1992

MEMORANDUM FOR RECORD

SUBJECT: Report of Dive Inspection of Burns Harbor Breakwater

1. A dive inspection was conducted of selected sections of Burns Harbor Breakwater on 24 June 192 by CERC divers. The purpose was to determine the distribution and placement of submerged armor units for comparison to that used in the physical models constructed by CERC in FY66 and FY92, and determine if substantial amounts of armor stone are buried by sediments beyond the visible toe of the structure. The dive team consisted of David McGehee, Chuck Mayers, Ray Townsend and David Ballard. The conditions were calm, visibility was approximately 10 ft. and the water temperature was in the low 50's. Three lakeside sections and the head section were inspected by David McGehee, and two harborside sections by Chuck Mayers.

2. Station 38 + 00, Lakeside - Armor stone placement is very random and porous with numerous voids large enough for a diver to penetrate to second layer. Armor extends to toe at -40 ft. A small amount of bedding stone is visible under the outermost armor units, but no appreciable amount extends beyond the toe. The lake bed is fine sand. One armor unit is partially buried in the sand where there is no bedding stone, but the majority of the unit is above the lake bed. There is no indication of additional buried armor.

3. Station 25 + 00, Lakeside - Armor stone placement is very random, though somewhat tighter packed than station 38 + 00. Armor extends to toe, where a distinct layer of bedding stone extends outward about 30 ft. No indication of buried armor stone.

4. Station 15 + 00, Lakeside - Armor placement is very random, and the slope of this section is significantly flatter than previous sections. Armor extends to -30 ft. where bedding stone begins and extends outward over 100 ft to the lake bottom at - 40 ft. Apparently, the bedding stone is about 10 ft thick at the toe.

5. Head section, Lakeside and Harborside - Armor stone and smaller, angular stone intermixed on a relatively steep slope extend to the lake bed at - 40 ft. No sign of bedding stone at toe. No indication of buried armor units or scour at toe.

6. Station 15 + 00, Harborside - Armor is randomly placed on relatively steep slope. Armor extends to mud bottom at -30 ft, and beyond under the mud. No bedding stone visible.

7. Station 35 + 00, Harborside - Armor is more uniformly placed on a steeper slope from the waterline to about -10 ft, then becomes more random and porous. Armor extends to and beyond the mud bottom at -20 ft. No sign of bedding stone.

8. Summary – In general, the armor placement on the inspected cross sections is at least as random as that utilized in the model studies. Though the placement in both models was not untypical of what was observed, there was more variability in arrangement and porosity in the prototype. There was no indication of a transition to angular stone at – 27 ft, as called for in the design and constructed in both models. Rather, armor units extended to the lake bed. Distribution of bedding stone was erratic, ranging from none to excessive. Armor stone can be assumed to extend beyond the toe, i.e., the intersection of the natural lakebed, to the original construction toe on the harborside, probably as a result of deposition of fine sediments over the years. There is no indication of significant stone buried in the sediments on the lakeside.

David D. McGehee
Research Hyd. Engineer

Appendix C
Calculation of Equivalent Settlement from 1967 to 1989(92)

The U.S. Army Engineer District, Chicago (NCC) conducted a stone stability analysis (Chapter 5, Volume II) to determine stone loss or gain along each breakwater segment. The NCC analysis considered the overall cross section from +14 at the crest to -30 ft low water datum (LWD). However, observations of some overlain survey profiles from 1967 (as-built) and 1989(92) (Volume II, Appendix 5A), revealed area differences below the -30-ft LWD level. The Coastal Engineering Research Center (CERC) conducted an additional analysis using the overlain survey profiles to determine the area of stone lost or gained below the -30-ft LWD level. The NCC analysis was combined with the CERC analysis to calculate equivalent settlement of the structure during the 25-year period since construction. This appendix provides a description and results of the CERC analysis and equivalent settlement calculations.

To determine equivalent settlement for each segment, total areal change ΔA for the entire breakwater cross section (+14 ft LWD to lake bottom) was calculated. Calculations of ΔA in the +14- to -30-ft LWD region used cross-sectional area means for each segment determined by NCC (Table 5-3, Chapter 5, Volume II) for 1967 and 1989(92).

To calculate ΔA for the area below the -30-ft LWD region, a separate analysis was conducted by CERC. Cross-section profiles of surveys, beginning with the as-built profiles and ending with 1989(92) survey profiles, were overlain by NCC to visually compare differences in each profile (Volume II, Appendix 5A). CERC enlarged cross-section profiles of Stations 0+00 through 57+00 for greater ease in the analysis. For Stations 0+00 through 6+00, 1987 profiles were used for this analysis because 1989(92) profiles were not available. All other areal comparisons were between the 1967 as-built and 1989(92) profiles.

Areal change below -30 ft LWD was determined visually using the enlarged profiles for each station, and was measured using a compensating

polar planimeter (a simple instrument designed to measure plane areas). Prior to the measurements, a calibration was conducted to determine a scale constant to be used for conversion of units (Vernier units to square feet). An arbitrary square was drawn on the profile scale and measured using the planimeter. The measurement procedure consisted of taking a start reading from the planimeter, then tracing the selected area five times. A second planimeter reading was taken, and the difference between the two readings was calculated and divided by 5 to determine the area in Vernier units. The area was then converted to square feet. All selected areas below -30 ft LWD (either above or below the as-built profile) were measured in this manner.

The increase in volume due to maintenance stone placement had to be accounted for. The measured volume change was converted to change in tonnage of armor stone (ΔT). ΔA calculations (square feet) for both the +14 to -30 ft LWD and below -30 ft LWD were converted to tons using Equation C1:

$$\Delta T = \frac{\Delta A L (0.59)(145)}{2000} \tag{C1}$$

where

ΔT = change in weight, tons

ΔA = change in area, sq ft

L = segment length, ft

0.59 = solids in the armor layer, percent

145 = stone density, pcf

Calculated ΔT per segment for +14- to -30-ft LWD and below -30-ft LWD are given in Table C1.

To obtain the total ΔT per segment for the overall cross section (+14-ft LWD to lake bottom), ΔT for the region +14- to -30-ft LWD was added to the below -30-ft LWD ΔT, and the actual amount of maintenance stone placed per segment was subtracted. Maintenance stone totals per segment (see Table C1) were calculated using tonnage data obtained from the "Burns Waterway Harbor, Indiana, Breakwater Major Rehabilitation Draft Evaluation Report" (March 1993). Total ΔT for each segment is also provided in Table C1.

Equivalent settlement was then calculated by converting ΔT for each breakwater segment to square feet (ΔA) and dividing that number by the design base width (211 ft) of the breakwater. Table C2 provides the settlement of the breakwater per segment using the procedure described above. In Table C2,

Table C1
Calculation of Areal Change from +14-ft LWD to Lake Bottom Per Breakwater Segment

Segment Number	Segment Length	ΔT (+14- to -30-ft LWD), tons	ΔT (Below -30-ft LWD), tons	Maintenance Stone, tons	1967 to 1989(92) Total ΔT, Tons
1	700	13,594[1]	-261[1]	18,045	-4,712[2]
2	1000	6,972	-766	27,294	-21,088
3	600	8,931	95	19,246	-10,220
4	900	14,706	1,600	27,471	-11,165
5	600	-4,440	27	8,483	-12,896
6	900	3,734	834	19,287	-14,719
7	400	-8,384	-685	3,627	-12,696
8	700	-19,912	-370	234	-20,516

[1] Determined from change in average area of cross-sectional region within the segment of interest based on: average area difference (1989(92) - 1967) × segment length × 0.59 (voids in the armor layer, percent) × 145 pcf (stone density)/2,000 pounds per ton.
[2] ΔT (Total) = {ΔT (+14- to -30-ft LWD) + ΔT (Below -30-ft LWD)} - Maintenance Stone.

Table C2
Calculation of Equivalent Settlement Per Breakwater Segment

Segment Number	Total ΔT, tons	ΔA, sq ft	Equivalent Settlement, ft
1	-4,712	-157[1]	-0.7
2	-21,088	-493	-2.3
3	-10,220	-398	-1.9
4	-11,165	-290	-1.4
5	-12,896	-502	-2.4
6	-14,719	-382	-1.8
7	-12,696	-742	-3.5
8	-20,516	-685	-3.2

[1] $\Delta A = (\Delta T \times 2000)/(0.59)(145)L$.

calculated settlement ranges from a minimum 0.7 ft in Segment 1 to a maximum 3.5 ft in Segment 7. The weighted average (by segment length) equivalent settlement is 2.1 ft.

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In 1984, Burns Harbor was nominated for inclusion in the Monitoring Completed Navigation Projects Program sponsored by Headquarters, U.S. Army Corps of Engineers. Burns Harbor was approved for monitoring in 1985 because it met both generic and site-specific selection criteria. This report summarizes the monitoring effort that took place at Burns Harbor. An overview of the monitoring effort is presented, included background information, a description of the monitoring effort, results of analyses, and conclusions that were drawn.

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